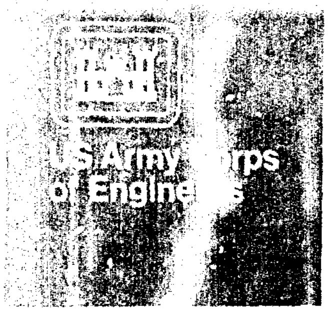




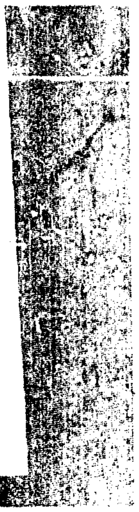

REPAIR, EVALUATION, MAINTENANCE, AND REHABILITATION RESEARCH PROGRAM

TECHNICAL REPORT REMR-CS-21

# IN SITU REPAIR OF DETERIORATED CONCRETE IN HYDRAULIC STRUCTURES: A FIELD STUDY

by

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April 1989

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

Under Civil Works Research Work Unit 32308

Monitored by Structures Laboratory
US Army Engineer Waterways Experiment Station
PO Box 631, Vicksburg, Mississippi 39180-0631

The following two letters used as part of the number designating technical reports of research published under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program identify the problem area under which the report was prepared:

| | Problem Area | | Problem Area |
|---|---|---|---|
| CS | Concrete and Steel Structures | EM | Electrical and Mechanical |
| GT | Geotechnical | EI | Environmental Impacts |
| HY | Hydraulics | OM | Operations Management |
| CO | Coastal | | |



COVER PHOTOS:

TOP — Network cracking in southeast corner of Pier No. 3, Dam 20, Mississippi River.

BOTTOM — Injection of downstream face of pier stem.

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

# REPORT DOCUMENTATION PAGE

Form Approved
OMB No 0704-0188
Exp Date Jun 30, 1986

| | |
|---|---|
| 1a. REPORT SECURITY CLASSIFICATION<br>Unclassified | 1b. RESTRICTIVE MARKINGS |
| 2a. SECURITY CLASSIFICATION AUTHORITY | 3. DISTRIBUTION/AVAILABILITY OF REPORT<br>Approved for public release; distribution unlimited. |
| 2b. DECLASSIFICATION/DOWNGRADING SCHEDULE | |
| 4. PERFORMING ORGANIZATION REPORT NUMBER(S)<br>BNL 42171 | 5. MONITORING ORGANIZATION REPORT NUMBER(S)<br>Technical Report REMR-CS-21 |
| 6a. NAME OF PERFORMING ORGANIZATION<br>Brookhaven National Laboratory<br>6b. OFFICE SYMBOL (If applicable) | 7a. NAME OF MONITORING ORGANIZATION<br>USAEWES<br>Structures Laboratory |
| 6c. ADDRESS (City, State, and ZIP Code)<br>Upton, NY 11973 | 7b. ADDRESS (City, State, and ZIP Code)<br>PO Box 631<br>Vicksburg, MS 39181-0631 |
| 8a. NAME OF FUNDING/SPONSORING ORGANIZATION<br>US Army Corps of Engineers<br>8b. OFFICE SYMBOL (If applicable) | 9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER |
| 8c. ADDRESS (City, State, and ZIP Code)<br>Washington, DC 20314-1000 | 10. SOURCE OF FUNDING NUMBERS<br>PROGRAM ELEMENT NO. \| PROJECT NO. \| TASK NO. \| WORK UNIT ACCESSION NO. 32308 |

11. TITLE (Include Security Classification)
In Situ Repair of Deteriorated Concrete in Hydraulic Structures: A Field Study

12. PERSONAL AUTHOR(S)
Webster, R. P., Kukacka, L. E., and Elling, D.

| 13a. TYPE OF REPORT | 13b. TIME COVERED | 14. DATE OF REPORT (Year, Month, Day) | 15. PAGE COUNT |
|---|---|---|---|
| Final Report | FROM ______ TO ______ | April 1989 | 67 |

16. SUPPLEMENTARY NOTATION This is a report of the Concrete and Steel Structures problem area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

17. COSATI CODES

| FIELD | GROUP | SUB-GROUP |
|---|---|---|
| | | |
| | | |

18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)

| | |
|---|---|
| Concrete | In situ repair |
| Cracking | Field test |
| Hydraulic structures | Pressure injection |

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

This report presents the results of a laboratory program directed toward developing and optimizing procedures for pressure injection repairs to be used in the field. Once developed, the procedures were evaluated in a small-scale field test at Lock and Dam 20 on the Mississippi River, Canton, MO. In general, the laboratory and field tests results demonstrate that pressure injection repair techniques can restore the integrity of cracked concrete hydraulic structures.

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|---|---|
| 20. DISTRIBUTION/AVAILABILITY OF ABSTRACT<br>☐ UNCLASSIFIED/UNLIMITED ☒ SAME AS RPT ☐ DTIC USERS | 21. ABSTRACT SECURITY CLASSIFICATION<br>Unclassified |
| 22a. NAME OF RESPONSIBLE INDIVIDUAL | 22b. TELEPHONE (Include Area Code) \| 22c. OFFICE SYMBOL |

DD FORM 1473, 84 MAR — 83 APR edition may be used until exhausted. All other editions are obsolete.

SECURITY CLASSIFICATION OF THIS PAGE
Unclassified

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# PREFACE

The study reported herein was authorized by Headquarters, US Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 32308, "In Situ Repair of Deteriorated Concrete," for which Mr. James E. McDonald, Research Civil Engineer, Concrete Technology Division (CTD), Structures Laboratory (SL), US Army Engineer Waterways Experiment Station (WES), is Principal Investigator. This work unit is part of the Concrete and Steel Structures Problem area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. The Overview Committee of HQUSACE for the REMR Research Program consists of Mr. James E. Crews and Dr. Tony C. Liu. Technical Monitor for this study was Dr. Liu.

This study was sponsored by WES and conducted by the Brookhaven National Laboratory (BNL) under the auspices of the Department of Energy under Support Agreement No. WESSC-86-01. This report was prepared by Messrs. R. P. Webster, L. E. Kukacka, and D. Elling, Process Sciences Division, BNL. The study was performed under the general supervision of Messrs. Bryant Mather, Chief, SL, and Kenneth L. Saucier, Chief, CTD. Direct supervision was provided by Mr. McDonald. Program Manager for REMR is Mr. William F. McCleese, CTD. This report was edited and prepared for publication by Mmes. Gilda Miller and Chris Habeeb, Editor and Editorial Assistant, respectively, Information Products Division, Information Technology Laboratory.

COL Dwayne G. Lee, EN, was Commander and Director of WES during the preparation of this report. Dr. Robert W. Whalin was Technical Director.

CONTENTS

## CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

| Multiply | By | To Obtain |
|---|---|---|
| centipoises | 0.001 | pascal seconds |
| cubic feet | 0.0283168 | cubic metres |
| Fahrenheit degrees | 5/9 | Celsius degrees or kelvins* |
| feet | 0.3048 | metres |
| gallons | 3.785 | litres |
| horsepower | 745.70 | watts |
| inches | 25.4 | millimetres |
| ounces (fluid) | 29.57 | millilitres |
| pounds | 453.5924 | grams |
| pounds (force) per square inch | 0.006894757 | megapascals |
| square feet | 0.0929030 | square metres |

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

# IN SITU REPAIR OF DETERIORATED CONCRETE IN HYDRAULIC STRUCTURES: A FIELD STUDY

## PART I: INTRODUCTION

### Background

1. Over the last 75 to 80 years, the use of portland-cement concrete in hydraulic structures, such as dams, spillways, lock chambers, and bridge support columns and piers, has been very extensive in the United States. The US Army Corps of Engineers estimates that it operates and maintains 536 dams and 260 lock chambers at 596 sites (Scanlon et al. 1983). More than 40 percent of these are more than 30 years old, and 29 percent were constructed before 1940. In addition, nearly one-half of the 260 lock chambers will reach their 50-year design lives by the turn of the century. Periodic inspections of these structures show that a large number of the older ones require significant maintenance, repair, and rehabilitation.

2. Repairs to many such structures involve the removal of the deteriorated concrete and replacement with new concrete to varying extents. Considerable savings in time and cost for the rehabilitation of highly deteriorated concrete structures could be realized if methods and materials were developed to repair them without extensive removal of the deteriorated concrete. To this end, Brookhaven National Laboratory (BNL), under contract to the US Army Corps of Engineers, has carried out a program entitled "In Situ Repair of Deteriorated Concrete in Hydraulic Structures." The results from Phase One and Phase Two of this program were documented in reports to the Corps of Engineers by Webster and Kukacka (1987, 1988).

3. The objectives of Phase One of the BNL program were to identify (a) the forms of deterioration most prevalent in concrete hydraulic structures and (b) existing methods and materials commonly used for the repair and rehabilitation of concrete structures. This information then was evaluated to determine the applicability of the various repair methods and materials for in situ repair.

4. According to a survey begun in 1982 by the US Army Engineer Waterways Experiment Station (USAEWES) (McDonald and Campbell 1985), the three most common problems encountered in the Corps' civil works concrete hydraulic

structures were (a) cracking, (b) seepage, and (c) spalling. These three problems accounted for 77 percent of the 10,096 deficiencies identified in a review of inspection reports. Concrete cracking was the most frequent and accounted for 38 percent of the total defects. In situ repair may not be readily applicable to problems of seepage; however, such procedures seem suited to repairing deterioration caused by cracking and spalling.

5. Three techniques for repairing cracks and two techniques for repairing spalled concrete were identified as being most applicable for in situ restoration. The methods include pressure injection, polymer impregnation, and the addition of reinforcement. In conjunction with these procedures, thin reinforced overlays and shotcrete were chosen as methods for repairing spalled concrete and resurfacing a cracked structure after it has been repaired. Based upon these findings, BNL developed a laboratory testing program in Phase Two to evaluate two of the crack repair methods: pressure injection and polymer impregnation.

6. The primary objectives of the Phase Two program were to experimentally evaluate and develop new methods and materials for the in situ repair of cracked concrete hydraulic structures by pressure injection and polymer impregnation. A laboratory-scale test program was developed to evaluate the effectiveness of (a) selected injection adhesives to repair air-dried and water-saturated cracked concrete and (b) polymer impregnation for repairing highly cracked concrete.

7. In general, the results of the Phase Two program indicated that pressure injection can effectively restore the integrity of air-dried and water-saturated cracked concrete. For example, concrete slabs that had pre-injection pulse velocities of 7,000 to 11,000 ft*/sec had pulse velocities of 13,500 to 15,000 ft/sec after injection. Sound, uncracked concrete normally has a pulse velocity of 14,000 to 15,000 ft/sec. The splitting tensile strength of air-dried concrete, repaired by pressure injection, varied between 410 and 845 psi, depending upon the adhesive used. Water-saturated concrete repaired by injection had splitting tensile strengths varying between 435 and 703 psi. Sound concrete had a splitting tensile strength of about 600 psi.

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

8. The laboratory results indicated that polymer impregnation can improve the quality of the concrete surrounding the crack network. However, its effectiveness in sealing the crack network depended on the viscosity of the impregnant used. The two methods, pressure injection and polymer impregnation, can be used in conjunction to effectively repair and improve the overall quality of the structure to be rehabilitated.

9. Based upon these results, BNL developed a program in Phase Three to develop and evaluate pressure injection procedures in the field.

## Phase Three Program Objectives

10. The emphasis of the Phase Three program was directed toward developing and optimizing pressure injection procedures to be used in the field. Once developed, the procedures were evaluated in a small-scale field test at Lock and Dam No. 20 on the Mississippi River, Canton, MO.

## PART II: LABORATORY TEST PROGRAM

11. The primary objectives of the laboratory test program were to develop and optimize procedures for pressure injection repair and to evaluate equipment to be used in the small-scale field test. This was accomplished in a series of large-scale laboratory tests using three highly cracked concrete shielding blocks, 8 ft 4 in. high by 5 ft 4 in. wide by 3 ft thick, with crack patterns similar to those observed in the pier stems at Lock and Dam No. 20, Canton, MO (Figures 1 and 2). Tests also were run on a 15-ft-high cracked concrete retaining wall. Maximum crack widths in the shielding blocks and retaining wall varied from 0.04 to 0.06 in.

12. The pressure injection repair procedure used in all of the tests follows:

- a. Measure preliminary ultrasonic pulse velocity.
- b. Sandblast the concrete to remove surface laitance.
- c. Clean the cracks with compressed air.
- d. Glue injection ports to the surface of the concrete.
- e. Coat the surface of the concrete with a gel epoxy to seal the surface of the cracks and prevent leakage of the injected adhesive.
- f. Measure preinjection ultrasonic pulse velocity.
- g. Inject the crack network with epoxy.
- h. Measure postinjection ultrasonic pulse velocity.
- i. Core the concrete and run splitting tensile strength tests on sections cut from the cores.

A discussion of each step is presented.

13. Preliminary measurements of ultrasonic pulse velocity were made with a portable Pundit Ultrasonic Tester (Figure 3). Readings were taken before starting repair work to obtain an accurate indication of the condition of each structure. All readings were taken with the indirect or surface transmission method.

14. The surface of each structure was sandblasted to remove any surface laitance that might interfere with the bonding of the injection ports or sealing of the surface of the cracks.

15. After sandblasting the surface of the concrete, compressed air was used to remove any debris that might interfere with injecting epoxy into the cracks.

Figure 1. Concrete shielding blocks

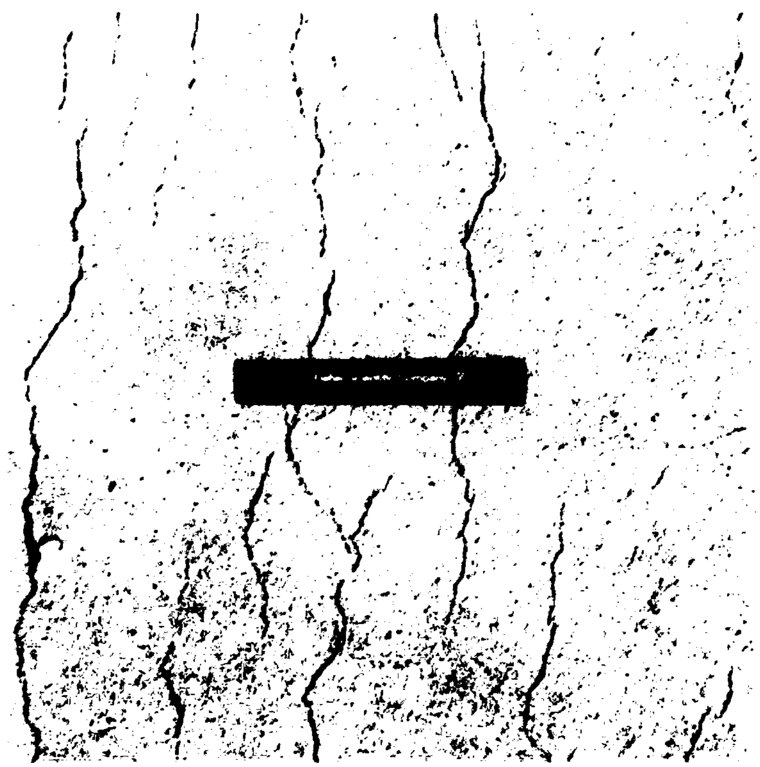

Figure 2. Typical crack pattern in the shielding blocks

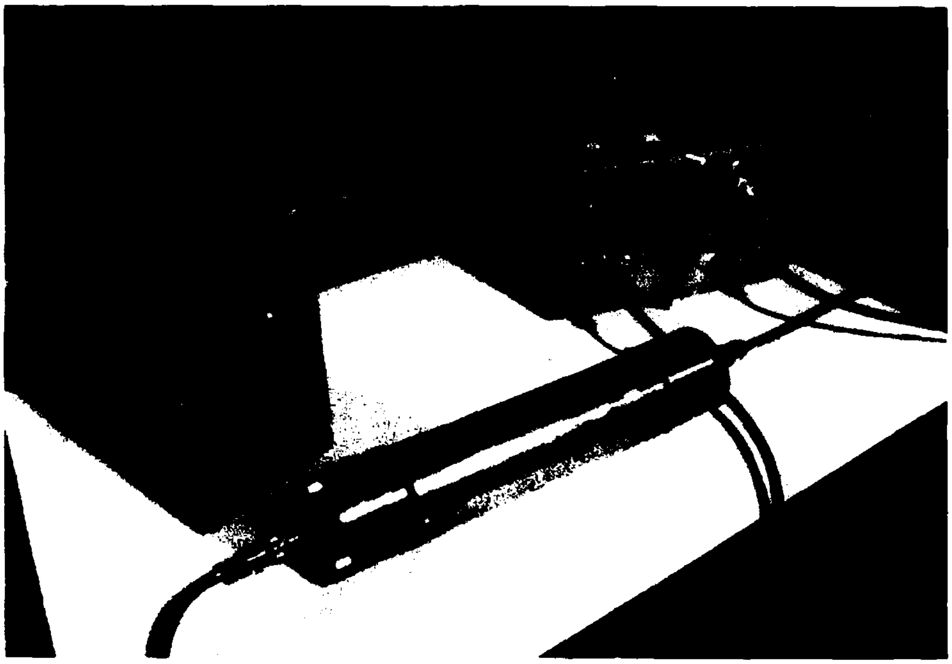

Figure 3. Pundit Ultrasonic Tester

16. The epoxy was injected into the crack network using injection ports glued over the surface of the cracks. The injection ports consisted of 1.5-in.-diam by 0.5-in.-thick wooden dowels with a 1/8-in. hole drilled into the center. Injection ports were randomly spaced by eye to ensure adequate coverage of the crack network. Some work was done using molded polyethylene injection ports; however, this was discontinued after several unsuccessful attempts to securely bond them to the surface of the concrete.

17. Immediately after the injection ports were placed, a 4-in.-wide area around each crack was sealed with a gel epoxy to prevent leakage of epoxy from the crack network during injection. The concrete surface was sealed with the same epoxy (Dural International, Inc.'s Duralcrete gel) used to place the injection ports to provide continuity in the seal coat. To completely seal the area around the injection ports, it was necessary to apply the epoxy seal coat before the epoxy used to glue the injection ports had gelled.

18. Once the epoxy seal coat was fully cured, a second set of ultrasonic pulse velocity measurements was taken to determine the influence of the seal coat on the readings.

19. The crack network then was injected with de Neef America, Inc.'s Denepox 40 epoxy, a very low viscosity (40 cP at 25° C), 100 percent solids,

two-component epoxy resin designed specifically for pressure injection. It has a resin-to-hardener mixing ratio of 3.33 to 1 by wt (2.85 to 1 by vol) and is relatively insensitive to the presence of moisture. Results of previously conducted laboratory tests (Webster and Kukacka 1988) have shown that water-saturated, cracked concrete repaired with Denepox 40 epoxy exhibits an average splitting tensile strength of 703 psi. Water-saturated, cracked concrete repaired using other various resins had splitting tensile strengths averaging 550 psi. Uncracked control concrete was measured to have an average splitting tensile strength of 616 psi.

20. The resin was injected into the crack network using Otto Engineering Inc.'s A3-10 epoxy injector. The A3-10 is a pneumatically operated, portable injection unit consisting of an aluminum suitcase, two stainless steel resin tanks, and two 20-ft dispensing lines which connect to a static mixing head immediately in front of the injection nozzle. The suitcase houses two positive displacement pumps, resin feed lines, and the pneumatic logic circuit which operate the unit. Mix ratios varying between 1:1 and 4:1 can be handled by the unit. The unit operates at an inlet air pressure and flow rate of 85 psi and 2 cfm, respectively, and dispenses the mixed epoxy at injection pressures up to 275 psi.

21. Injection of the crack network began at the lowest point and proceeded upward along the length of each crack. As connection was made with each port, evidenced by the pumping of epoxy out of the port, the ports were sealed by inserting a wooden peg into the hole in the center of the port. Although the injection pressures for this test were not measured, it is estimated that they varied between 150 and 275 psi.

22. Postinjection ultrasonic pulse velocity measurements were taken when the injection resin had sufficient time to fully cure. These measurements were then compared with the two sets of preinjection measurements to obtain an indication of the success of the injection.

23. Evaluation of the repair work also was based upon the visual examination of cores removed from the repaired areas and upon the results of splitting tensile strength tests [American Society for Testing and Materials (ASTM) C496 (1985)] performed on sections cut from the cores.

24. Preliminary ultrasonic pulse velocity data for the shielding blocks varied between 6,891 and 12,616 ft/sec and averaged 9,582 ft/sec. After injection with the Denepox 40 epoxy (Figure 4), the pulse velocity data varied

Figure 4. Pressure injection repair of concrete shielding block

between 8,079 and 13,731 ft/sec and averaged 11,228 ft/sec, representing an average increase of 17 percent in the pulse velocity.

25. Upon completion of the injection repair work, fifteen 3-in.-diam cores were removed from the blocks to better evaluate the success of the repairs. Visual examination indicated that with the exception of several cracks in two cores, all of the cracks appeared to be completely filled with epoxy. The cracks varied in thickness up to 0.04 in. and extended 3.5 to 4 in. below the surface of the block. Values for splitting tensile strength varied between 338 and 815 psi and averaged 608 psi, compared to an average of 546 psi for the cores taken from areas which appeared to be uncracked.

26. A large-scale laboratory test also was conducted on a cracked section of a 15-ft-high concrete retaining wall (Figures 5 and 6) to evaluate the equipment to be used in the field test. Before this test, the A3-10 epoxy injection equipment was used just as it had been received from the manufacturer and was operated with laboratory compressed air cylinders. To use the injection equipment in the field, it was modified to be supplied from a 2-HP, 20-gal portable air compressor.

27. The modifications involved adding a regulator/filter/dryer unit to the inlet air system to clean and dry incoming air from the compressor. Also,

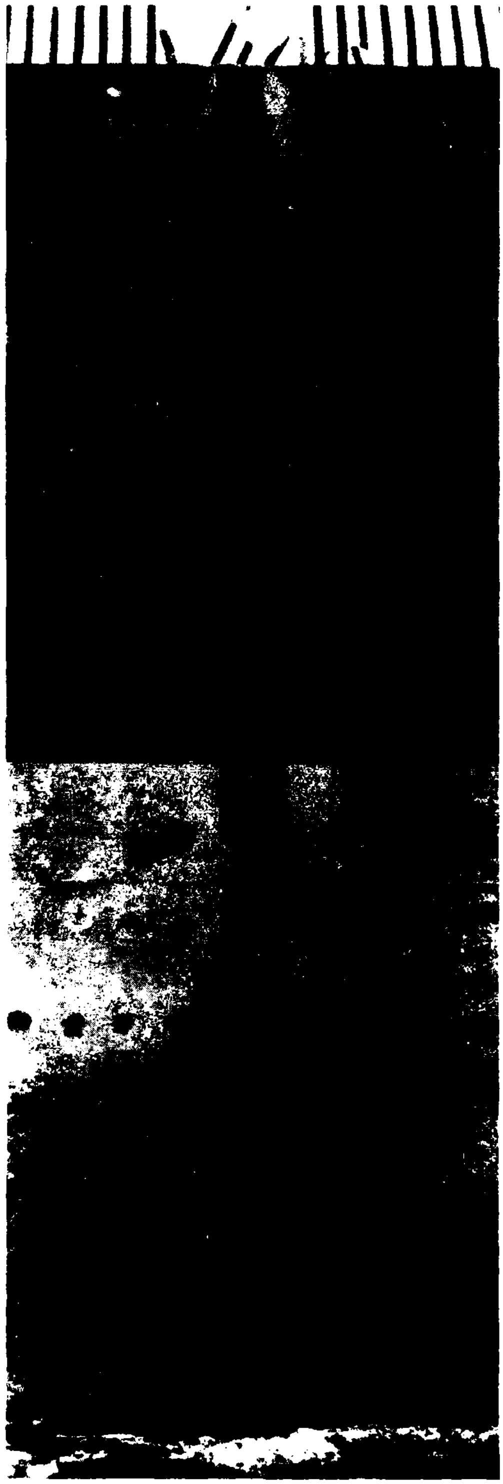

Figure 5. Concrete retaining wall repaired by epoxy injection

Figure 6. Pressure injection repair of concrete retaining wall

check valves were installed in the material output lines before the mixing head to prevent accidental back pressuring and mixing of the A and B components of the injection epoxy.

28. The crack in the retaining wall was sealed using the Duralcrete gel epoxy. In this test, the wooden dowel injection ports were placed approximately 9 in. on center along the 15-ft length of the crack.

29. The crack was injected, at a pressure of approximately 150 psi, with the Denepox 40 epoxy, beginning at the base of the wall and proceeding upward. With only two or three exceptions, the epoxy was successfully pumped up the wall from one injection port to the next. Injection took approximately 45 min.

30. Evaluation of the repair work was based upon ultrasonic pulse velocity measurements and visual examination and mechanical testing of cores removed from the wall.

31. Preliminary ultrasonic pulse velocities taken across the crack varied between 8,300 and 10,170 ft/sec and averaged 9,065 ft/sec. Ultrasonic pulse velocities in uncracked sections of the wall averaged 13,710 ft/sec. Postinjection ultrasonic pulse velocities varied between 10,990 and 14,220 ft/sec and averaged 13,180 ft/sec. This value represents a 45-percent

increase over the average preliminary pulse velocity. Also the repaired wall section has an average pulse velocity of 96 percent of that measured for uncracked sections.

32. Visual examination of the 3-in.-diam cores removed from the wall after repair showed that the crack extended a minimum of 12 in. into the wall (Figure 7). Crack thickness varied up to 0.04 in. The cracks appeared to contain 80- and 100-percent epoxy.

33. Results of splitting tensile strength tests varied between 591 and 805 psi and averaged 693 psi, compared to an average of 659 psi for the uncracked controls.

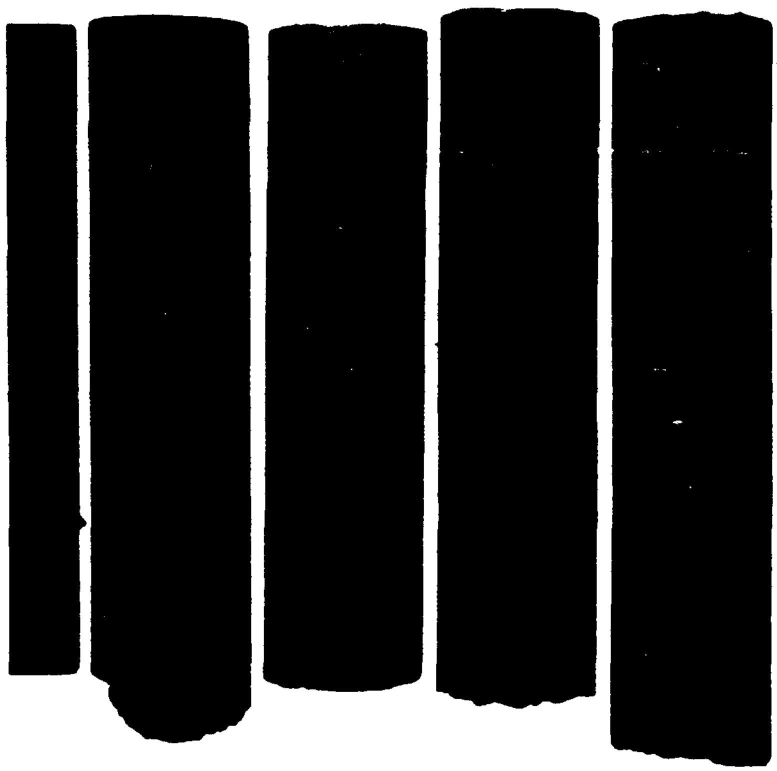

Figure 7. Cores removed from concrete retaining wall after repair

## PART III: FIELD TEST PROGRAM

### Background

34. After completion of the large-scale laboratory tests, a small-scale field test was performed on Pier No. 27 at Lock and Dam No. 20, Canton, MO, 17-25 Aug 1987 (Figure 8). The objectives were to demonstrate, under field conditions, the procedures developed in the laboratory and to evaluate the effectiveness of the materials and equipment selected for use.

35. Lock and Dam No. 20 is located at river mile 343.2 on the Mississippi River, Canton, MO. The structure, operated by the US Army Corps of Engineers (USACE), was placed into service in 1936 and is a part of the 9-ft Channel Navigation Project. Over the years, periodic inspections revealed cracks in many of the 42 concrete pier stems that support the dam service bridge. The condition of the concrete ranges from good (high compressive strength and no deterioration) to severely deteriorated (D-cracking, leaching, drummy with a loss of strength) (US Army Engineer District, Rock Island 1985).

Figure 8. Pier No. 27 (in foreground)

36. Damage to the pier stems is most significant in the top 11 ft, the area from el 490 to the top of the pier at el 501.* The damage is attributed to stresses developed by a lack of slip between the service bridge sole plates and the anchored bearing seats in the tops of the stems. Water allowed to pond because of the recessed bearing seats gains access to the pier stem interior through cracks caused by the anchor forces and accelerates deterioration of the concrete. Cracking in this area of the pier stems has been observed since 1939.

37. The repair work on Pier No. 27 was limited to the top 4.2 ft of the pier stem, i.e., that portion of the pier stem located above the archway ceiling of the walk-through area. The deterioration in this area was characterized by two major cracks that extended from the top of the pier stem down to the ceiling of the archway. Also, a network of cracks was visible on the upstream and downstream faces of the pier stem (Figures 9-14).

## Repair Procedure

38. The procedure used to repair Pier No. 27 is outlined and discussed in detail.

a. Erect scaffolding.

b. Measure preliminary ultrasonic pulse velocity.

c. Sandblast pier stem.

d. Place injection ports and seal pier stem surfaces.

e. Measure preinjection ultrasonic pulse velocity.

f. Inject crack network.

g. Measure postinjection ultrasonic pulse velocity.

h. Petrographic and mechanical analysis of cores.

### Erect scaffolding

39. Two 30-in.-wide by 12-ft-long wooden scaffolds were erected along the east and west faces of the pier stem to provide access to these sides. Each scaffold consisted of a 2- by 6-in. pine frame covered with 1/2-in. plywood. The scaffolding was suspended from the steel I-beams that support the service bridge, using 3/8-in.-diam wire rope attached to a 3-in. steel

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* All elevations (el) cited herein are in feet referred to National Geodetic Vertical Datum (NGVD) of 1929.

Figure 9. Downstream face of Pier No. 27

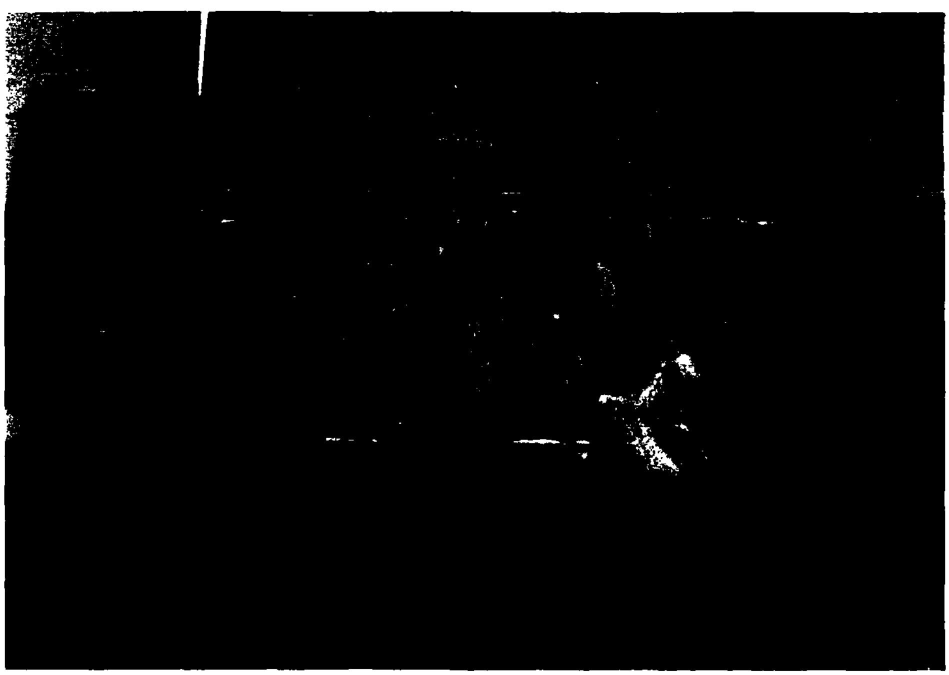

Figure 10. Upstream face of Pier No. 27

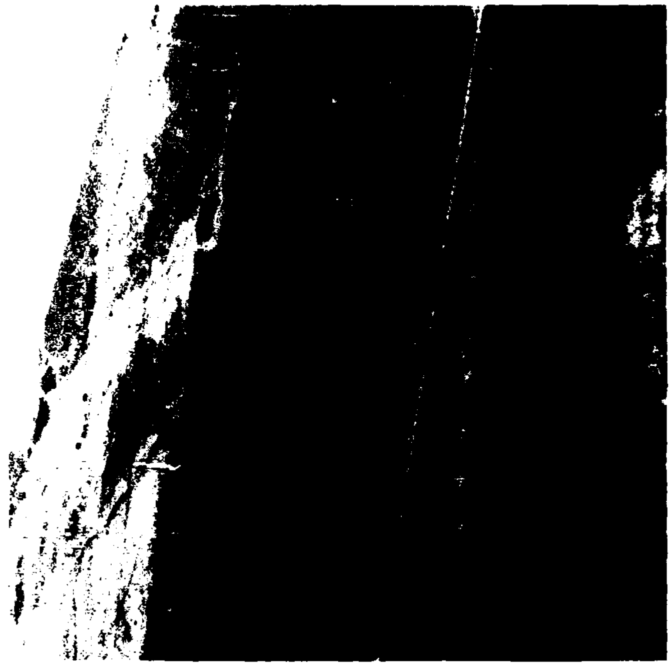

Figure 11. Network cracking in downstream corner of the Illinois face of Pier No. 27

Figure 12. Cracking in upstream corner of the Missouri face of Pier No. 27

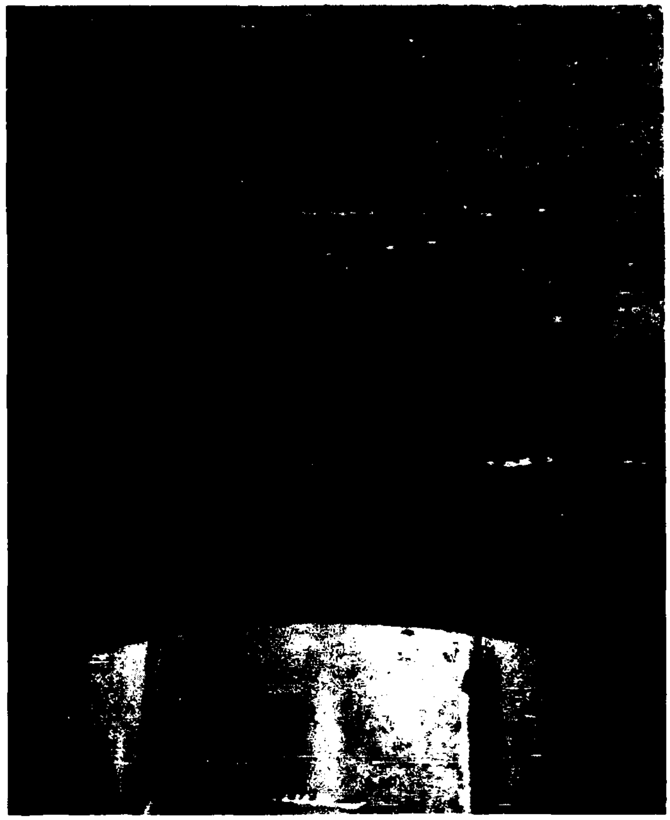

Figure 13. Major crack in downstream face of the stairwell

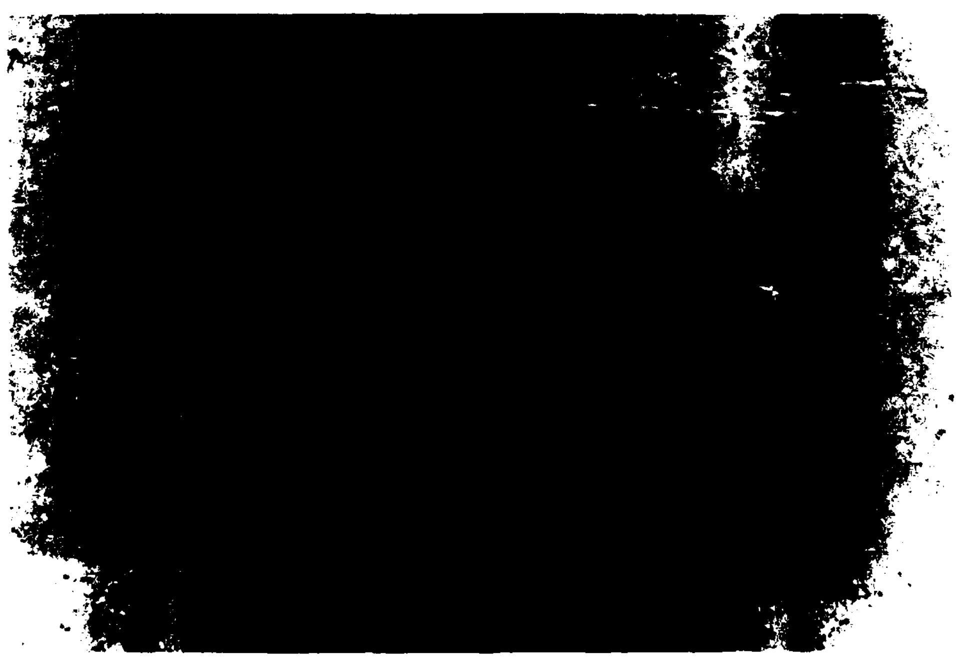

Figure 14. Major crack in upstream archway ceiling

channel by 1/2-in.-diam wrought steel eye bolts. The channels were held in place against the bottom flange of the I-beams using 4-in. "C" clamps (Figure 15).

40. Access to the upstream and downstream faces was provided by an existing work platform suspended over the sides of the dam from the service bridge crane (Figure 16).

Preliminary ultrasonic pulse velocity data

41. Preliminary ultrasonic pulse velocity data were taken between opposite points at six locations on the Illinois and Missouri faces of the pier stem using the direct transmission method. Readings also were taken between selected points on the faces of the stairwell, the top of the pier stem, the upstream face of the pier stem and the archway ceiling using the indirect transmission method. However, these readings were used only to supplement the data obtained from the direct transmission method, since the indirect method generally is considered less accurate than the direct method.* Primary emphasis was based upon the pulse velocity values obtained using the direct transmission method. The general locations of the pulse velocity readings are illustrated in Figure 17, and the readings are summarized in Table 1.

42. Preliminary pulse velocity values are obtained for only 1 of the 6 locations at which direct transmission readings were taken. That location (No. 4) had a pulse velocity of 9,717 ft/sec. Preliminary data was obtained for 8 of the 11 locations at which supplemental readings were taken using the indirect transmission method. These pulse velocity values varied between 5,357 and 8,333 ft/sec and averaged 7,085 ft/sec. Concrete that has an ultrasonic pulse velocity of 12,000 to 15,000 ft/sec is generally classified as being in good condition, 10,000 to 12,000 ft/sec in questionable condition, and 7,000 to 10,000 ft/sec in poor condition (Muenow 1966).

Sandblasting of pier stem

43. Once the preliminary ultrasonic pulse velocity data were collected, all faces in the top 4.5 ft of the pier stem were sandblasted to remove paint,

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* Pundit Manual for Use with the Portable Ultrasonic Non-Destructive Digital Indicating Tester, C.N.S. Instruments Ltd., 61-63 Holmes Road, London, NW5, England.

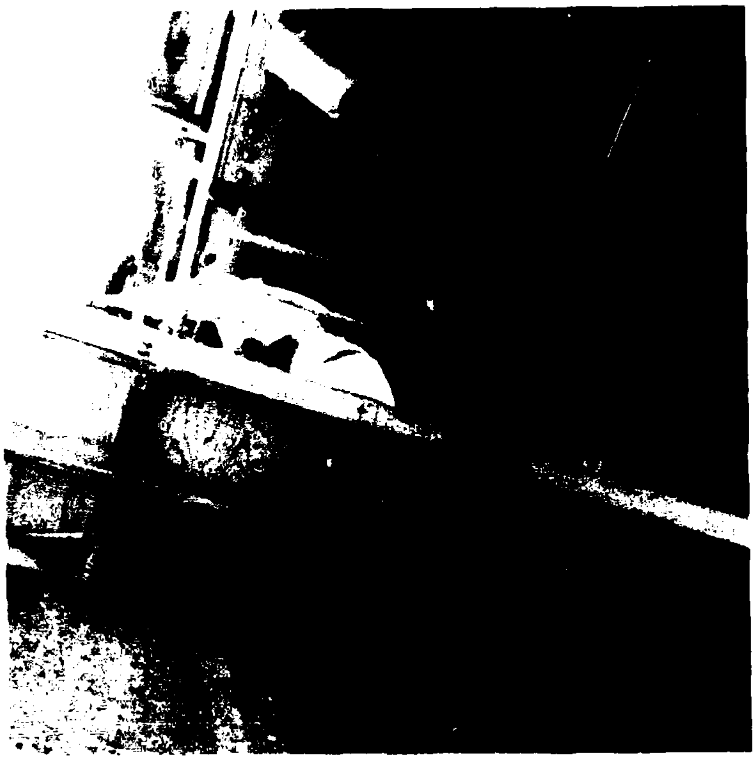

Figure 15. View from below scaffolding suspended from service bridge

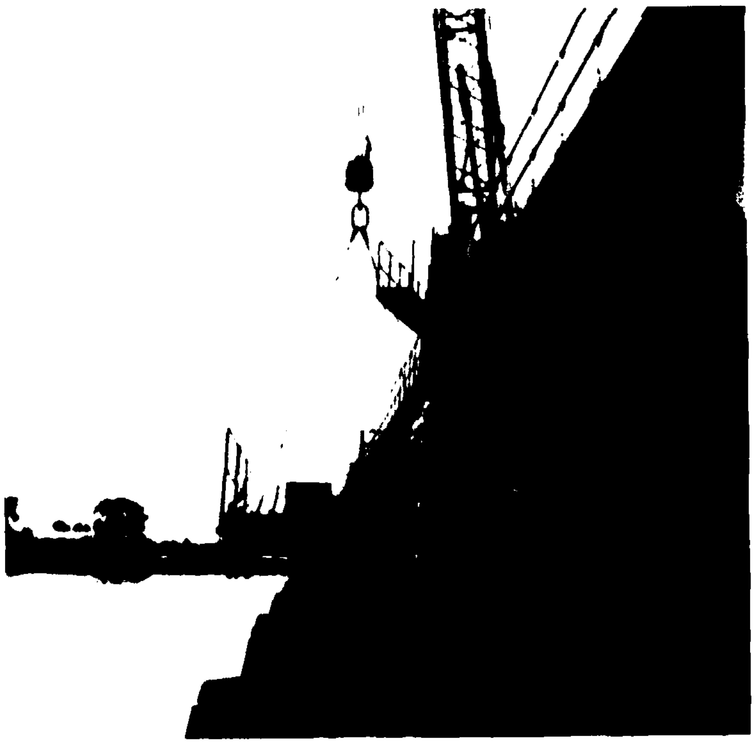

Figure 16. Work platform suspended from service bridge crane

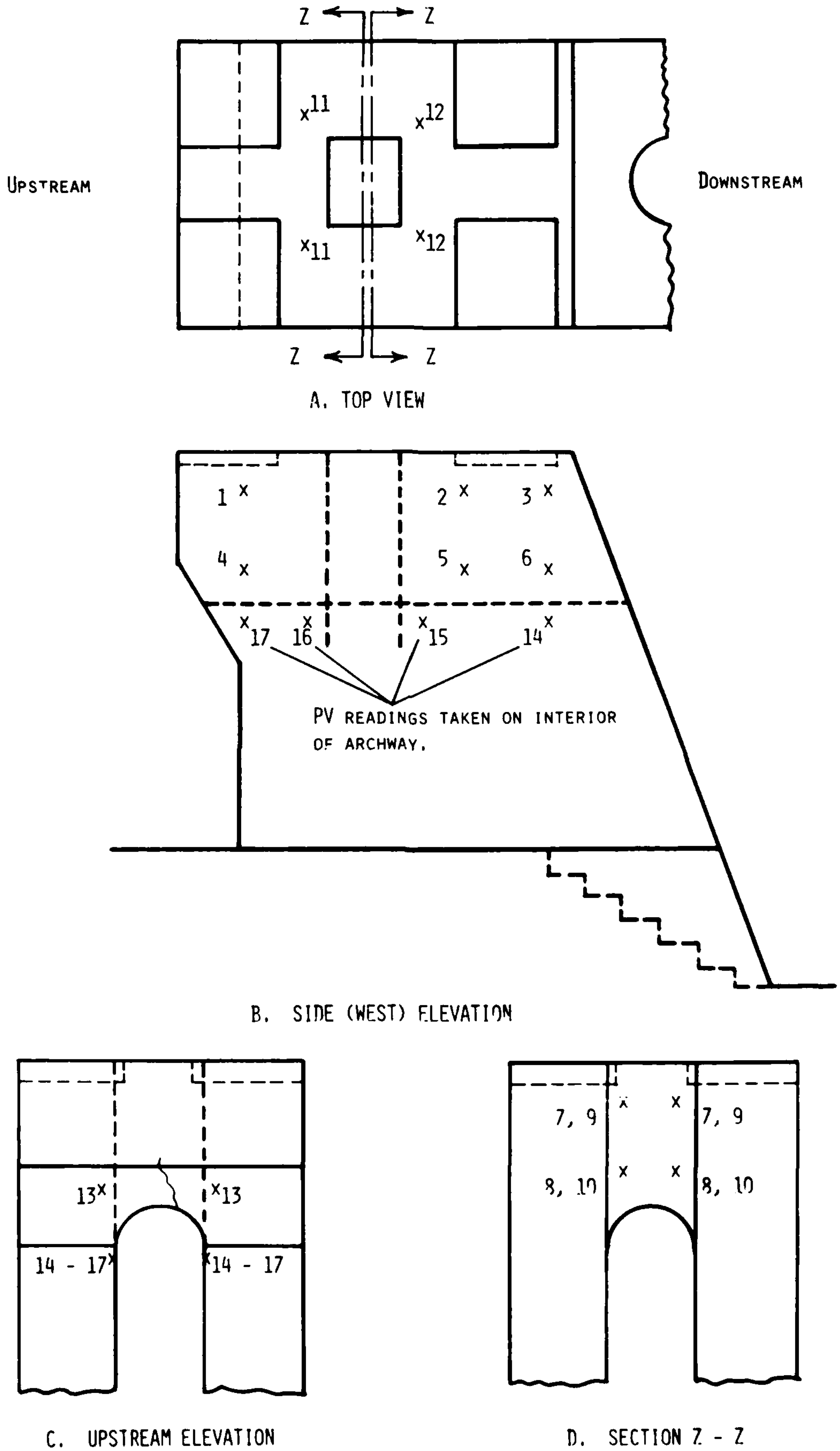


Figure 17. Locations of ultrasonic pulse velocity measurements

Table 1

Ultrasonic Pulse Velocity Data

| Location/No. | Type of Transmission | Ultrasonic Pulse Velocity, ft/sec — Preliminary* | Pre-Injection** | Post-Injection |
|---|---|---|---|---|
| Side of pier stem | Direct | | | |
| 1 | | NR+ | 7,275 | 11,555 |
| 2 | | NR | NR | 6,075 |
| 3 | | NR | NR | 5,215 |
| 4 | | 9,717 | 12,243 | 15,124 |
| 5 | | NR | NR | 14,984 |
| 6 | | NR | NR | 7,835 |
| Downstream face of stairwell | Indirect | | | |
| 7 | | 7,075 | 5,190 | 8,152 |
| 8 | | 5,357 | 9,375 | 14,019 |
| Upstream face of stairwell | Indirect | | | |
| 9 | | 8,333 | 8,174 | 13,043 |
| 10 | | 7,537 | 11,236 | 14,851 |
| Top of pier stem | Indirect | | | |
| 11 | | NR | NR | 7,936 |
| 12 | | NR | NR | 6,133 |
| Upstream face | Indirect | | | |
| 13 | | NR | NR | 8,452 |
| Archway ceiling | Indirect | | | |
| 14 | | 6,910 | 4,790 | 7,184 |
| 15 | | 7,065 | 5,625 | 8,272 |
| 16 | | 7,082 | 6,879 | 8,098 |
| 17 | | 7,320 | 5,523 | 8,402 |

* Readings taken before sealing the surface of the pier stem.
** Readings taken after sealing the surface of the pier stem.
\+ NR = unable to obtain a pulse time reading.

efflorescence, and other impurities that might interfere with the bonding of the injection ports and sealing of the surface.

44. The pier stem was sandblasted using a small portable unit (Sandy Jet Pressure Blaster, Model F-110) (Figure 18), which was operated off a 100-cfm portable air compressor. A total of 640 lb of No. 460 sandblasting sand (94-percent passing a No. 30 sieve) cleaned a surface area of approximately 350 sq ft.

Sealing of pier stem

45. After sandblasting, the injection ports were placed, and sealing of the surface of the pier stem was started. The injection ports consisted of 1-1/2-in.-diam by 1/2-in.-thick wooden dowels, each with a 1/8-in.-diam hole drilled through the center. Ports were randomly spaced to ensure adequate coverage of the crack network (Figure 19). About 140 ports were placed on the surface of the pier stem.

46. The ports were glued to the surface using Dural International, Inc.'s Duralcrete gel epoxy. This is a two-part, nonsag, high modulus adhesive that is intended for vertical and overhead repairs of concrete. This epoxy also was used to seal the surface of the pier stem.

47. The injection ports were attached to the surface of the pier stem by first placing a bead of epoxy around the perimeter of the port using a syringe, centering the port over the crack, and then applying epoxy with a paint brush to seal the concrete around the port. This work was done in small sections to ensure that the ports could be placed and the surrounding concrete sealed before the epoxy began to gel. A total of 4-1/2 gal of Duralcrete gel was used to place the injection ports and seal the pier stem.

48. The cracks around the bridge seats and those underneath the steel superstructure that could not be reached by hand were sealed by pouring Dural's Flexolith epoxy into the area. Flexolith is a relatively low viscosity, flexible, low-modulus epoxy intended for use in overlays and patching. Approximately 1 gal of material was used to seal the bridge seats.

Preinjection ultrasonic pulse velocity data

49. After the epoxy seal coat had fully cured, preinjection ultrasonic pulse velocities were measured to determine if the seal coat had any effect on the preliminary readings taken before work on the pier stem had begun. These results are summarized in Table 1.

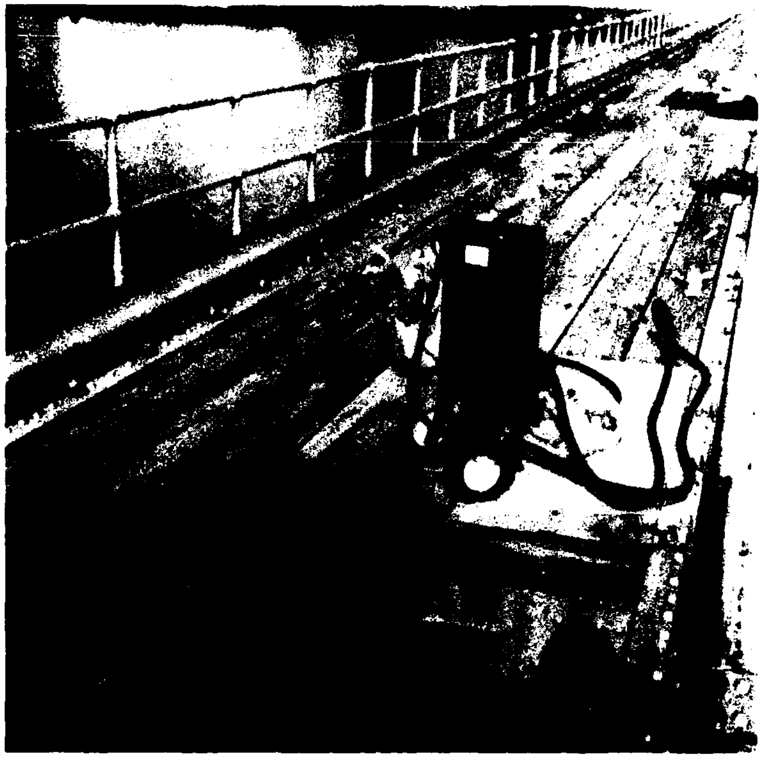

Figure 18. Portable sandblasting unit

Figure 19. Placement of injection ports on surface of pier stem

50. Preinjection pulse velocities were obtained for 2 of the 6 locations at which direct transmission readings were taken. These locations, No. 1 and 4, had pulse velocity values of 7,275 and 12,243 ft/sec, respectively. Preinjection pulse velocities were also obtained for 8 of the 11 locations at which supplemental readings were taken using the indirect transmission method. The values varied between 5,190 and 11,236 ft/sec and averaged 7,099 ft/sec.

51. It will be noted that a comparison of the preliminary and preinjection pulse velocity values obtained using the indirect transmission method indicates that six of the eight readings decreased after the pier stem had been sealed. This is contrary to what is expected, and a definite explanation for it is not readily available. However, the weaknesses of this method of measurement may have partially contributed to the reductions noted in the readings.

## Pier stem injection

52. The injection resin used to repair the pier stem was de Neef America, Inc., Denepox 40 epoxy. Denepox 40 is an ultra-low viscosity (40 cP at 25° C), two-component epoxy designed specifically for pressure injection repairs.

53. The epoxy was injected into the pier stem using a modified version of an Otto Engineering, Inc., A3-10 portable epoxy injection machine (Figure 20). Modifications made to the A3-10 were discussed in Part II of this report. The injection machine was operated off a 2-HP, 20-gal portable air compressor (Figure 21).

54. All workers in the immediate vicinity of the repair were required to wear protective jump suits with hoods (disposable), rubber gloves, and a full face shield during the injection process.

55. The pier stem was injected in sections and required 2 days to complete. The sections were injected in the following sequence:

Day 1: center section of downstream face
east corner of downstream face
downstream corner of east (Illinois) face
west corner of downstream face
downstream corner of west (Missouri) face
downstream face of stairwell and archway ceiling
west face of stairwell

Figure 20. A3-10 portable epoxy injection machine

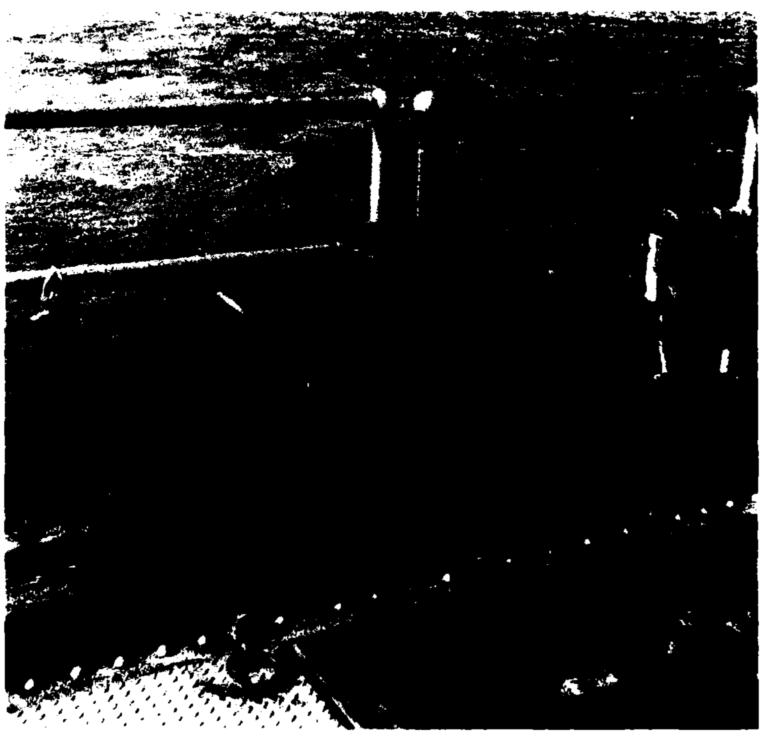

Figure 21. Portable air compressor used to operate the A3-10 injection machine

east face of stairwell
downstream half of pier stem top

Day 2: center section of upstream face
west side of upstream face
upstream corner of west face
east side of upstream face
upstream corner of east face
upstream face of stairwell and archway ceiling
upstream half of pier stem top

56. Injection of the pier stem began on the downstream face along the major crack that extended from the top of the pier stem down to the archway ceiling (Figures 22 and 23). Injection was started at the second port from the top of the crack: within 2 to 3 min epoxy appeared in the ports located directly above and below. These ports were plugged, and injection of resin into the starting port was continued until epoxy was observed coming out of ports farther along the crack. The starting port was plugged and the operation moved to another port along the crack. Injection proceeded downward along the length of the crack until all connecting cracks were filled, as evidenced by the pumping of epoxy out of the injection ports. All unconnected cracks remaining in the section were then injected. The operation then was moved to the next section.

57. The general technique used for each section involved first injecting all major cracks and then injecting any remaining cracks that did not connect with the larger ones. This technique was particularly successful on the eastern corner of the downstream face, the upstream and downstream faces of the stairwell, and the archway ceiling. These were the sections that contained the largest number of interconnected cracks and required the greatest amount of epoxy to fill them.

58. The eastern corner of the downstream face contained an extensive network of interconnected cracks. A majority of these cracks were filled by injecting epoxy into the centermost injection port over a 45-min period.

59. The major cracks that ran through the upstream and downstream faces of the stairwell and archway ceiling were filled by injecting epoxy into one injection port located at the bottom edge of each stairwell face. From these ports epoxy could be pumped along the entire length of the crack in the archway ceiling and up the entire length of the crack in the face of the

Figure 22. Injection of downstream face of pier stem

Figure 23. Close-up of injection nozzle in injection port

stairwell. This procedure was necessary because the shape and size of the stairwell limited access to only a few injection ports.

60. Some sections, such as the west side of the downstream face and the upstream ends of the east and west faces contained very few interconnected cracks. Therefore it was necessary to inject resin into each port.

61. Periodically, 50-ml gel samples of the epoxy were taken to ensure that the proper mixing ratio was maintained and that the epoxy was curing properly. Samples generally were collected once every 45 to 60 min and whenever work was begun on a new section. Only three samples did not cure within 24 hr. These samples were taken while injecting the upstream ends of the east and west faces and were a bit gummy after 24 hr; they cured after approximately 2 weeks.

62. Approximately 5 gal of Denepox 40 epoxy was injected into the top of the pier stem. Several problems were encountered during injection, but the operation went well.

63. The problems encountered were related primarily to two areas: mechanical operation of the injection equipment and leakage of epoxy from around the injection ports or through the seal coat.

64. The A3-10 injection equipment broke down several times necessitating emergency field repairs. In fact, the equipment had to be operated manually for about the first 2 hr during the second day. The equipment also broke down at the end of the operation, and attempts to restart it were unsuccessful. As a result, two isolated cracks were never injected with epoxy, one of which was located in the upstream face of the stairwell and the other in the upstream archway ceiling.

65. There were problems in maintaining an adequate seal around the injection ports to prevent leakage of epoxy during injection. Work was stopped several times because of this problem. The leakage may have been the result of (a) a weakness or opening in the epoxy around the injection port, (b) back pressure developing because the opening in the port was not centered over the crack, (c) back pressure developing because the crack had been filled with epoxy, or (d) the epoxy being injected into small, isolated cracks at too high a pressure. It is estimated that the injection pressure at the nozzle varied between 170 and 250 psi.

### Postinjection ultrasonic pulse velocity data

66. Postinjection ultrasonic pulse velocity data were taken approximately 19 hr after the injection was completed. At this point only three gel samples had not fully cured. The pulse velocities are summarized in Table 1. In general, postinjection pulse velocities were obtained at each of the 17 selected locations.

67. The greatest improvement in the integrity of the pier stem can be seen by comparing the preliminary and postinjection pulse velocities measured between the Illinois and Missouri faces of the pier stem, using the direct transmission technique. Preliminary pulse velocities were detectable at only one of the six locations on the sides of the pier stem. After injection, however, velocities were obtained for each location. The postinjection velocities varied between 5,215 and 15,124 ft/sec and averaged 10,131 ft sec. While the average pulse velocity indicates that the quality of the concrete in the pier stem may be classified as "questionable," it must be remembered that prior to injection the integrity of the pier stem was so poor that it was possible to obtain a pulse velocity value for only one of the selected locations. In addition, it should be noted that the injection process should help to slow down future deterioration due to the fact that the surfaces of the cracks have been sealed, thereby preventing penetration of water into those cracks.

68. A close examination of the pulse velocity values measured between the Illinois and Missouri faces indicates that the greatest improvement in the integrity of the pier stem occurred in the upstream half, i.e., that portion between the upstream face and the stairwell. This is illustrated by the high postinjection pulse velocity values measured for locations No. 1 and 4: 11,555 and 15,124 ft/sec, respectively. The significant increases noted in the supplemental pulse velocity values measured on the upstream face of the stairwell, locations No. 9 and 10, also help to support this conclusion.

69. The generally low postinjection pulse velocity values measured in the downstream half of the pier stem would appear to indicate that either the crack network in that portion of the pier stem was only partially repaired or that there are cracks within the interior of the pier stem which were not repaired at all.

### Concrete cores

70. Approximately 3 weeks after completion of the injection work, seven

4-in.-diam cores and one 6-in.-diam core were removed from Pier No. 27 by personnel from the US Army Engineer District, Rock Island. The core locations are shown in Figure 24, and the drilling logs are presented in Appendix A.

71. Four of the cores were tested by the Rock Island District. Cores DS-3, US-1, and MO-1 were tested in compression and averaged 6,630 psi. Ultrasonic pulse velocities for these cores varied between 11,088 and 12,578 ft/sec and averaged 12,055 ft/sec. Core Top-2 was examined petrographically (Appendix B). The four remaining cores (DS-1, DS-2, IL-1, and Top 1) and a section of core US-1 were sent to BNL for splitting tensile strength tests and testing of resistance to cycles of freezing and thawing (Figures 25 and 26).

72. A visual examination of these cores showed that the crack network in three of them, cores DS-1, DS-2, and Top 1, were 80- to 90-percent filled with epoxy. Crack widths in these cores varied between 0.002 and 0.050 in. Core IL-1 appeared to contain no epoxy. The section of core US-1 contained no visible cracks.

73. Ultrasonic pulse velocity tests were done to evaluate the integrity of each of the cores. Discs were then cut from each core, and their splitting tensile strength was determined. The results of these tests are summarized in Table 2.

74. The ultrasonic pulse velocities of the three cores in which the crack network contained 80- to 90-percent epoxy (cores DS-1, DS-2, and Top-1) varied between 10,338 and 13,192 ft/sec and averaged 11,759 ft/sec. These test results are consistent with the pulse velocities measured for cores DS-3, US-1, and MO-1 by the Corps of Engineers. The pulse velocity measured for core IL-1, which was highly cracked and contained no visible epoxy in the crack network, was 7,166 ft/sec. The pulse velocity of the uncracked section of core US-1 was 13,988 ft/sec.

75. Splitting tensile strengths for cores DS-1, DS-2, and Top-1 varied between 414 and 700 psi and averaged 513 psi. This figure represents a 48-percent increase over that measured for core IL-1. The splitting tensile strength of the uncracked core of US-1 was 548 psi.

76. Tests were also run to evaluate the resistance of the repaired and nonrepaired cores to cycles of freezing and thawing. The cores were subjected to a total of 100 cycles of freezing and thawing in accordance with ASTM C 666, Procedure A (ASTM 1984). Evaluation of the cores was based upon

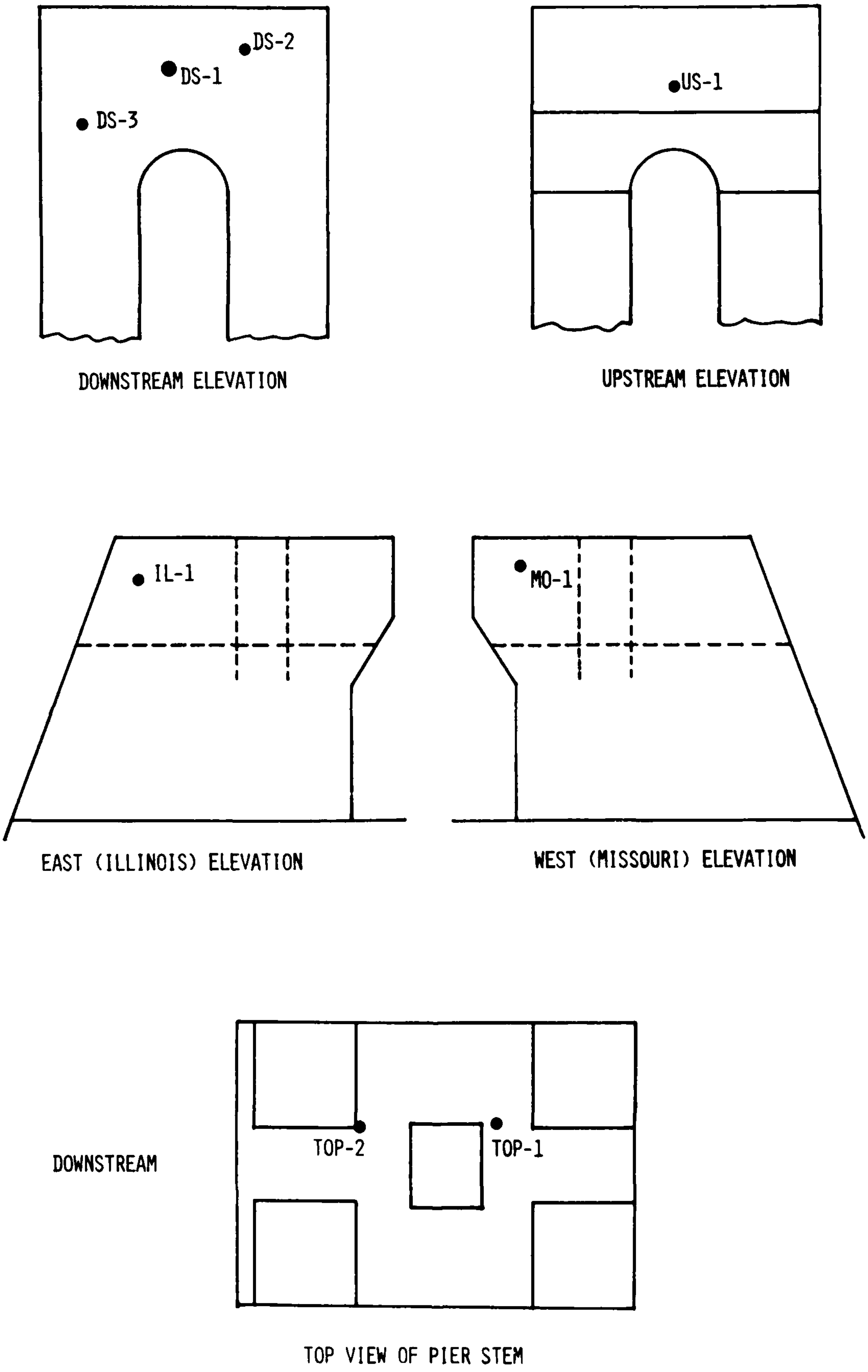


Figure 24. Locations of concrete cores

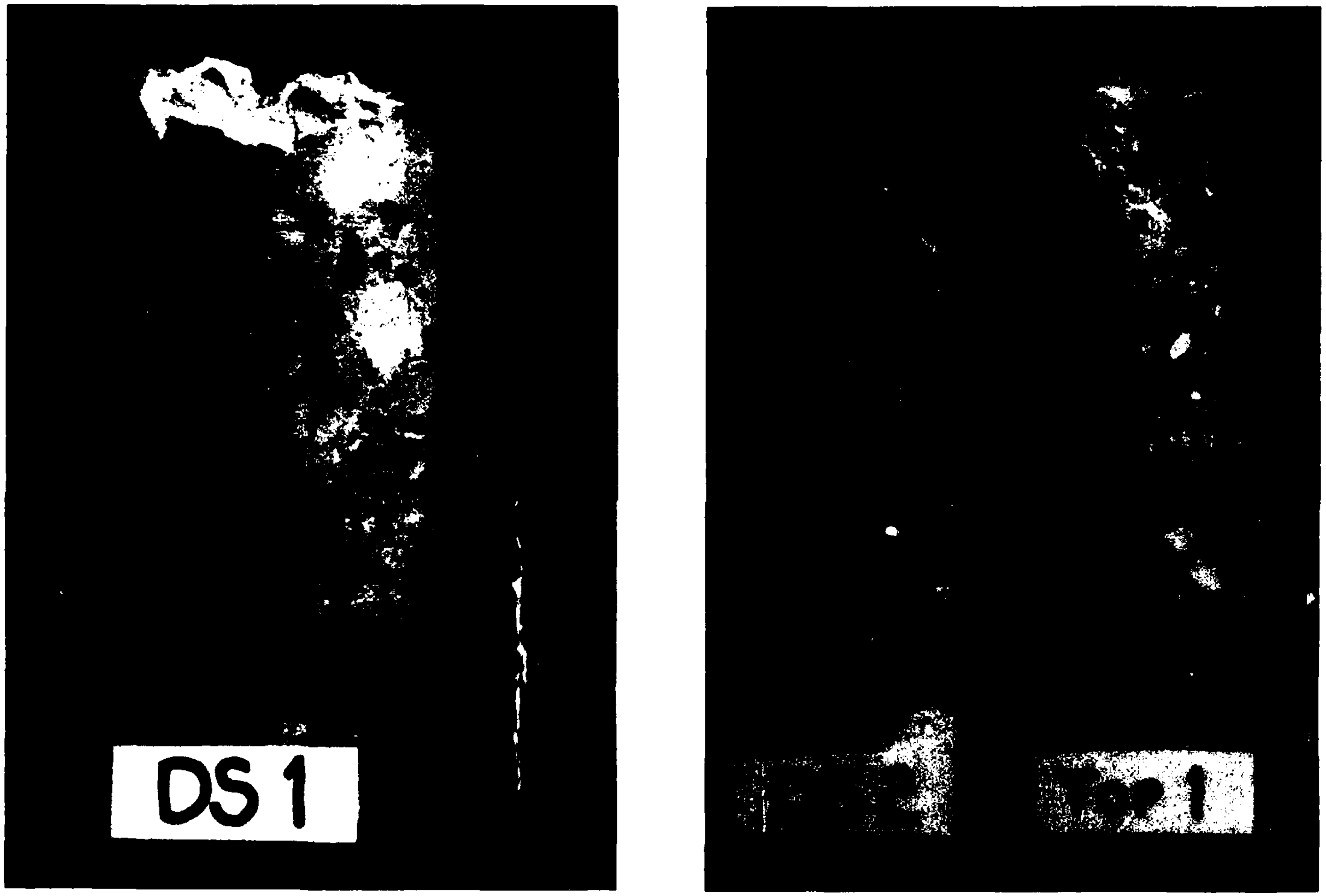


Figure 25. Cores taken from the downstream face (DS-1 and DS-2) and top (Top 1) of Pier No. 27

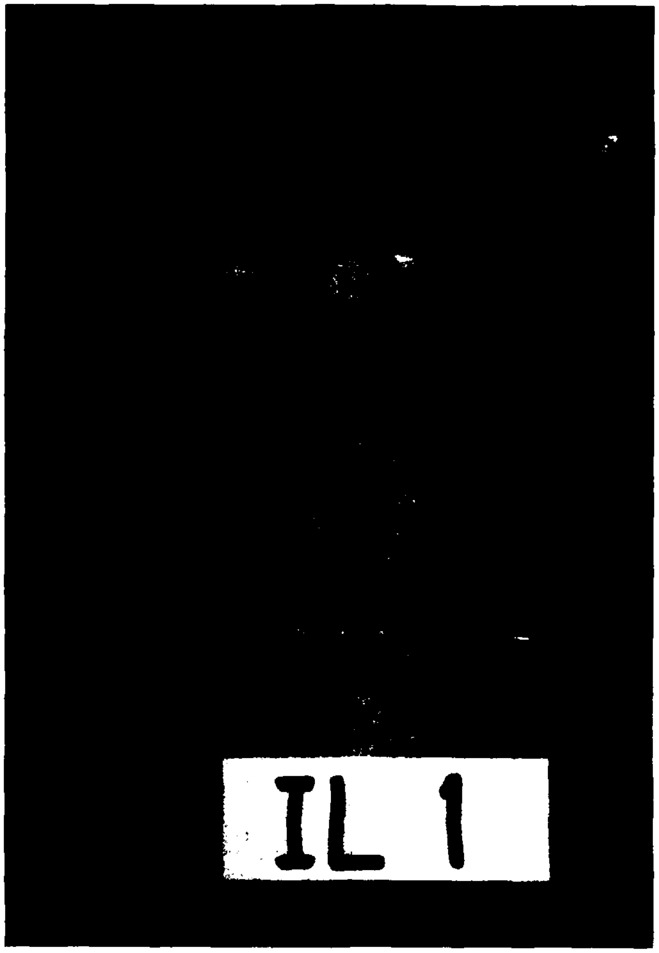


Figure 26. Core removed for Pier No. 27
containing no visible epoxy

ultrasonic pulse velocity, splitting tensile strength test data, and visual examination. Test results are summarized in Table 3.

77. Ultrasonic pulse velocity data were taken throughout the duration of the test. The results indicate that the pulse velocities measured for cores Top-1 and DS-2 decreased approximately 26 percent after being subjected to 100 cycles of freezing and thawing while the pulse velocity of core IL-1 decreased about 64 percent. Initial pulse velocities for cores Top-1, DS-2, and IL-1 were 13,092, 12,439, and 10,417 ft/sec, respectively. After 100 cycles of freezing and thawing, these cores had pulse velocities of 9,867, 8,927, and 3,800, respectively.

78. Splitting tensile strength tests were run on discs cut from the cores after 25 and 100 cycles. Discs were not cut from IL-1 after 25 cycles due to its short length. Results indicated that the average strength of the

Table 2

Ultrasonic Pulse Velocity and Splitting Tensile Strength Test Data for Cores Removed from Pier No. 27

| Core | Ultrasonic Pulse Velocity, ft/sec | Splitting Tensile Strength, psi | Comments |
|---|---|---|---|
| DS-1 | 13,192 | 414 | Cracks filled 80 to 90% with epoxy |
| DS-2 | 11,748 | 426 | Cracks filled 80 to 90% with epoxy |
| Top-1 | 10,338 | 700 | Cracks filled 80 to 90% with epoxy |
| US-1 | 13,988 | 548 | Core uncracked |
| IL-1 | 7,166 | 346 | Cracks contain no epoxy |

Table 3

Ultrasonic Pulse Velocity and Splitting Tensile Strength Test Data (before and after freeze-thaw testing)

| | Core | | |
|---|---|---|---|
| | TOP-1 | DS-2 | IL-1 |
| Initial Data: | | | |
| Pulse velocity, ft/sec | 13,092 | 12,439 | 10,417 |
| Splitting tensile strength, psi | 700 | 426 | 346 |
| After 25 cycles: | | | |
| Pulse velocity ft/sec | 11,967 | 10,670 | 6,897 |
| Splitting tensile strength, psi | 593 | 330 | NT* |
| After 50 cycles: | | | |
| Pulse velocity, ft/sec | 10,725 | 11,095 | 5,978 |
| After 75 cycles: | | | |
| Pulse velocity, ft/sec | 10,422 | 10,356 | 5,274 |
| After 100 cycles: | | | |
| Pulse velocity, ft/sec | 9,867 | 8,927 | 3,800 |
| Splitting tensile strength, psi | 380 | 393 | 79 |

* NT = no test.

repaired cores (Top-1 and DS-2) had decreased from 563 to 462 psi after 25 cycles of freezing and thawing and from 563 to 387 psi after 100 cycles. The splitting tensile strength IL-1 decreased from 346 to 79 psi after 100 cycles.

79. In spite of the reductions noted in the pulse velocities and splitting tensile strengths of the repaired cores (Top-1 and DS-2), the cores appeared to be in good condition. A visual examination of the cores after 100 cycles of freezing and thawing indicated only minor surface scaling and no deterioration was noted around the repaired cracks. However, IL-1 exhibited severe deterioration in the form of cracking, aggregate popouts, and erosion of the cement paste. It was initially believed that the crack network in IL-1 contained no epoxy. However, examination of the interior of the core after testing indicated that some of the cracks contained 20- to 30-percent epoxy. This helps explain why the core withstood so many freeze-thaw cycles. It was anticipated that core IL-1 would begin to deteriorate sooner than it did since the nonair-entrained concrete was highly cracked to begin with.

## Economic Evaluation

80. Based upon the experience gained during the small-scale field test, a preliminary economic projection was made to determine the cost of repairing the top 4.2 ft of 1, 5, and 10 pier stems. For purposes of the evaluation, the following assumptions were made:

- a. The design and dimension of the pier stem are similar to those at Lock and Dam No. 20.
- b. The repair work uses equipment and techniques similar to those used in the field test.
- c. The work is done in the central region of the United States.
- d. Quality control evaluation, such as ultrasonic pulse velocity testing and coring, is done by personnel from the Corps of Engineers.

81. Table 4 summarizes the manpower requirements to repair a typical pier stem. These figures are based upon the time taken to perform these tasks during the field test. Approximately 184 man-hr are required to repair one pier stem. In projects where several pier stems are being repaired, this figure should decrease with the efficient use of personnel and as workers become familiar with the repeated requirements of the job. However, the

reduction in manpower requirements due to "the learning curve" were not factored into the economic analysis.

82. The projected costs for repairing 1, 5, and 10 pier stems by epoxy injection are summarized in Table 5. The costs vary between $24,535 for repairing 1 pier stem to $85,630 for repairing 10. Equipment represents 58 percent of the costs involved in repairing 1 pier stem. The significance of the equipment costs, however, is reduced as more pier stems are repaired.

83. Labor costs become more significant as a larger number of pier stems are repaired. Labor represents 17 percent of the costs to repair 1 pier stem and 49 percent of the costs to repair 10.

84. Materials and supplies account for 4.3 to 7.0 percent of the total costs of repair. Some of these costs increase directly with the number of piers being repaired, such as the costs of injection epoxy, seal coat epoxy, and sandblasting sand. Other costs, such as that for scaffolding, increase slightly, as many of these items can be reused.

Table 4

Estimated Man-Power Requirements for the Repair of a Typical Pier Stem

| Task | Manpower Requirements | | |
| --- | --- | --- | --- |
| | Type | Number | Man-hr |
| Erection of scaffolding | Laborer | 3 | 24 |
| | Crane operator | 1 | 4 |
| Sandblast pier stem | Laborer | 2 | 16 |
| | Crane operator | 1 | 3 |
| Placement of injection ports and sealing of pier stem | Laborer | 2 | 28 |
| | Crane operator | 1 | 5 |
| Injection of pier stem | Laborer | 3 | 48 |
| | Crane operator | 1 | 16 |
| Removal of injection ports and seal coat | Laborer | 2 | 32 |
| | Crane operator | 1 | 8 |
| Total | Laborer | | 148 |
| | Crane operator | | 36 |

Table 5

Analysis of Costs of Epoxy Injection Repair

| Parameters | Number of Pier Stems Repaired 1 | 5 | 10 |
|---|---|---|---|
| Equipment costs (EC) | | | |
| Air compressors (100 cfm & 2 cfm units) | $ 7,500 | $ 7,500 | $ 8,000* |
| Portable sandblasting unit | 500 | 500 | 500 |
| Pressure injection equipment | 5,800 | 5,800 | 11,600 |
| Misc. equipment and tools | 500 | 500 | 500 |
| Total EC | $14,300 | $14,300 | $20,600 |
| Materials and supplies (MS) | | | |
| Injection epoxy (Denepox 40) | $ 225 | $ 1,125 | $ 2,250 |
| Seal coat epoxy (Duralcrete gel) | 185 | 925 | 1,850 |
| Sandblasting sand | 60 | 300 | 600 |
| Cleaning solvent | 18 | 36 | 54 |
| Scaffolding | 300 | 600 | 900 |
| General supplies | 275 | 325 | 400 |
| Total costs of MS | $1,063 | $3,311 | $ 6,054 |
| Labor (L)** | | | |
| Laborers, 148 man-hr/stem @ $18.95/hr | $ 2,805 | $14,025 | $28,050 |
| Crane operator, 36 man-hr/stem @ 25.12/hr | 904 | 4,520 | 9,040 |
| Supervisory labor (15% of total operating labor) | 556 | 2,782 | 5,564 |
| | $ 4,265 | $21,327 | $42,654 |
| Total repair costs (TRC) = EC + MS + L | $19,628 | $38,938 | $69,308 |
| Overhead and profit (25% of TRC) | $ 4,907 | $ 9,735 | $17,327 |
| Total project costs | | | |
| TRC + overhead and profit | 24,535 | $48,673 | $86,635 |

* Purchase a second 2-cfm air compressor at $500 and a second pressure injection machine.

** Laborer and crane operator wage rates were obtained from Engineer News-Record, 17 Sep 1987, and are an average for St. Louis, MO, and Chicago, IL.

## PART IV: SUMMARY AND RECOMMENDATIONS

85. According to a survey by the USAEWES (McDonald and Campbell 1985), the three most common problems encountered in the Corps' civil works concrete hydraulic structures were (a) cracking, (b) seepage, and (c) spalling. These three problems accounted for 77 percent of the 10,096 deficiencies identified during a review of inspection reports. Concrete cracking was observed most often, accounting for 38 percent of the total deficiencies. While in situ procedures may not be readily applicable to repair seepage, they apparently are suited to repairing deterioration caused by cracking and spalling.

86. Brookhaven National Laboratory, under contract to the USACE, is conducting a program to experimentally evaluate and develop new methods and materials for the in situ repair of cracked concrete hydraulic structures. The major emphasis of this work was the evaluation of techniques for pressure injection repair. The results of Phase One and Phase Two of this program were documented in reports to the Corps (Webster and Kukacka 1987, 1988). The results of Phase Three are presented in this report.

87. The emphasis in Phase Three was directed toward the development and optimization of pressure injection procedures to be used in the field. Once developed, the procedures were evaluated in a small-scale field test at Lock and Dam 20, Canton, MO.

88. The laboratory phase of the program was concerned with the development and optimization of the repair techniques to be used in the field. A series of large-scale laboratory tests were conducted using three highly cracked, 8-ft 4-in.-high by 5-ft 4-in.-wide by 3-ft-thick concrete shielding blocks and a 15-ft-high cracked concrete retaining wall.

89. The pressure injection repair procedure used in all the tests was:

- a. Measure preliminary pulse velocity.
- b. Sandblast the concrete to remove surface laitance.
- c. Clean the cracks with compressed air.
- d. Glue injection ports to the surface of the concrete.
- e. Coat the surface of the concrete with a gel epoxy to seal the surface of the cracks and prevent leakage of the injection adhesive.
- f. Inject a low viscosity, water-compatible epoxy into the cracks.
- g. Measure postinjection pulse velocity.

h. Core the concrete and measure the splitting tensile strength of sections cut from the cores.

90. Results of the tests on the concrete shielding blocks indicated that the preliminary ultrasonic pulse velocities of the blocks varied between 6,891 and 12,626 ft/sec and averaged 9,582 ft/sec. Postinjection velocities varied between 8,079 and 13,731 ft/sec and averaged 11,228 ft/sec. Splitting tensile strength tests on discs cut from 3-in.-diam cores and removed from the repaired blocks averaged 608 psi, compared to an average of 546 psi for the uncracked controls.

91. Similar results were obtained for the repair work performed on the concrete retaining wall. Preliminary ultrasonic pulse velocities varied between 8,300 and 10,170 ft/sec and averaged 9,065 ft/sec, while postinjection ultrasonic pulse velocities varied between 10,990 and 14,220 ft/sec and averaged 13,180 ft/sec. Splitting tensile strength tests showed the repaired wall with an average strength of 693 psi. Uncracked sections of the wall had an average splitting tensile strength of 659 psi.

92. When the large-scale laboratory tests were completed, a small-scale field test was performed on Pier No. 27, Lock and Dam No. 20, Canton, MO, on 17-25 Aug 1987. The objectives of the field test were to demonstrate, under field conditions, the procedures developed in the laboratory and to evaluate the effectiveness of the materials and equipment selected for use.

93. The repair work on Pier No. 27 was limited to the top 4.2 ft of the pier stem, or that portion located above the archway ceiling of the walk-through area. The deterioration in this area was characterized by two major cracks that extended from the top of the pier stem down to the ceiling of the archway. Also, a network of cracks was visible on the upstream and downstream faces of the pier stem.

94. Approximately 5 gal of Denepox 40 epoxy was injected into the top of the pier stem over a period of 2 days. A number of problems were encountered during injection, but the job, in general, was a success. Postinjection pulse velocity values measured at six locations between the Illinois and Missouri faces of the pier stem varied between 5,215 and 15,124 ft/sec and averaged 10,131 ft/sec. Before injection, it was possible to obtain a pulse velocity value at only one of these locations. That location had a preliminary pulse velocity of 9,717 ft/sec and a postinjection pulse velocity of 15,121 ft/sec.

95. A visual examination of three cores taken from Pier No. 27 after completion of the injection repair work indicated that 80 to 90 percent of the crack network within these cores appeared to be filled with epoxy. Ultrasonic pulse velocity tests indicated that these cores had an average pulse velocity of 11,759 ft/sec, as compared to a value of 7,166 ft/sec that was measured for a highly cracked core containing no visable epoxy within its crack network. The splitting tensile strength of the repaired cores averaged 513 psi compared to a value of 548 psi that was measured for an uncracked control.

96. Tests to evaluate the resistance of repaired cores to deterioration due to cycles of freezing and thawing indicated a 26-percent decrease in the pulse velocity values and a 31-percent decrease in the splitting tensile strength after being subjected to 100 cycles.

97. An economic analysis of the repair procedure projects that the cost of repairing the top 4.2 ft of a pier stem by epoxy injection varies between $24,535 for repairing 1 pier stem to $85,630 for repairing 10.

98. Based upon the general success of the small-scale field test, it has been demonstrated that cracked concrete hydraulic structures can be repaired in situ by pressure injection. However, a number of areas still should be optimized, as shown by the problems encountered in the field. It is therefore recommended that additional work be done to continue the optimization of the pressure injection repair techniques developed in Phases Two and Three. Suggested areas of research include:

a. Identification and development of a better method for attaching the injection ports to the concrete. Excessive leakage of epoxy from around the injection ports was encountered on several occasions during the field test.

b. Evaluation of the need to drill into the crack network to facilitate the penetration of epoxy into the interior of the network. Ultrasonic pulse velocities data indicated that some areas within the interior of the pier stem may not have been completely filled with epoxy.

c. Evaluation of additional materials for use as sealants. A number of problems were encountered when epoxy injected into the crack network caused the epoxy gel coat seal to bubble and leak during injection.

d. A field demonstration of the optimized repair process.

99. It is recommended that future work also include:

a. Laboratory studies to evaluate the durability characteristics, such as resistance to cycles of freezing and thawing of

air-dried and water-saturated cracked concrete that was repaired by injection.

b. The compilation of information pertaining to the health, safety, and environmental effects of the various chemicals (i.e., adhesives and solvents) used in the repair process.

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# APPENDIX A

# CONCRETE CORE DRILLING LOGS

Hole No. DS-1

| DRILLING LOG | DIVISION North Central | INSTALLATION NCRED-G | SHEET OF SHEETS |
|---|---|---|---|

| | |
|---|---|
| 1. PROJECT Dam 20 | 10. SIZE AND TYPE OF BIT 6" Thinwall |
| 2. LOCATION (Coordinates or Station) Pier 27, Downstream Face. | 11. DATUM FOR ELEVATION SHOWN (TBM or MSL) |
| 3. DRILLING AGENCY | 12. MANUFACTURER'S DESIGNATION OF DRILL Truco |
| 4. HOLE NO. (As shown on drawing title and file number) DS-1 | 13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED — UNDISTURBED |
| 5. NAME OF DRILLER Jerry Wickersham | 14. TOTAL NUMBER CORE BOXES |
| 6. DIRECTION OF HOLE ☐ VERTICAL ☐ INCLINED Horizontal DEG. FROM VERT. | 15. ELEVATION GROUND WATER |
| | 16. DATE HOLE — STARTED — COMPLETED Sep 87 |
| 7. THICKNESS OF OVERBURDEN | 17. ELEVATION TOP OF HOLE |
| 8. DEPTH DRILLED INTO ROCK | 18. TOTAL CORE RECOVERY FOR BORING % |
| 9. TOTAL DEPTH OF HOLE 1.8' | 19. SIGNATURE OF INSPECTOR |

| ELEVATION a | DEPTH b | LEGEND c | CLASSIFICATION OF MATERIALS (Description) d | % CORE RECOVERY e | BOX OR SAMPLE NO. f | REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g |
|---|---|---|---|---|---|---|
| 0.25 | | | Thin epoxy coating on surface. Several vertical, horizontal and angled cracks. Only one discontinuous crack is not filled with epoxy. Cracks as narrow as .0025 inch are filled with epoxy. | | | |
| 0.50 | | | | | | |
| 0.75 | | | | | | |
| 1.00 | 1.0 | | Broken to retrieve core at 1.0 ft. Shear is through coarse aggregate. Epoxy visible on shear surface | | | |
| | 1.1 | | | | | |
| 1.25 | | Elect. Conduit | Electrical conduit, 1½ inch I.D. encountered from 1.1 to 1.55 ft. Rebar #4 encountered at 1.6 ft. | | | |
| 1.50 | | 1.55 | | | | |
| 1.75 | 1.8 | Rebar | Occasional discontinuous angled crack from 1.0 to 1.8 ft. No epoxy noted. | | | |
| | | | Broken to retrieve core at 1.8 ft. Shear is around and through coarse aggregate. No epoxy noted on shear surface | | | |
| | | | See general description of concrete | | | |

Hole No. DS-2

**DRILLING LOG**

DIVISION: North Central

INSTALLATION: NCRED-G

SHEET OF SHEETS

1. PROJECT: Dam 20
2. LOCATION (Coordinates or Station): Pier 27, Downstream Face
3. DRILLING AGENCY:
4. HOLE NO. (As shown on drawing title and file number): DS-2
5. NAME OF DRILLER: Jerry Wickersham
6. DIRECTION OF HOLE: Horizontal ☐ VERTICAL ☐ INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN
8. DEPTH DRILLED INTO ROCK
9. TOTAL DEPTH OF HOLE: 1.0
10. SIZE AND TYPE OF BIT: 4" Thinwall
11. DATUM FOR ELEVATION SHOWN (TBM or MSL)
12. MANUFACTURER'S DESIGNATION OF DRILL: Truco
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED | UNDISTURBED
14. TOTAL NUMBER CORE BOXES
15. ELEVATION GROUND WATER
16. DATE HOLE: STARTED | COMPLETED Sep 87
17. ELEVATION TOP OF HOLE
18. TOTAL CORE RECOVERY FOR BORING %
19. SIGNATURE OF INSPECTOR

| ELEVATION a | DEPTH b | LEGEND c | CLASSIFICATION OF MATERIALS (Description) d | % CORE RECOVERY e | BOX OR SAMPLE NO. f | REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g |
|---|---|---|---|---|---|---|
| | 0.1 | | Thin epoxy coating on surface | | | |
| 0.25 | | 0.25 | Angled crack from 0.1 to 0.25 ft. is about .007 in. wide and appears to be completely filled with epoxy. | | | |
| 0.50 | 0.4 | 0.6 | The angled crack from 0.4 to 0.6 ft. is about .03 in. wide and appears to be completely filled with epoxy. There are occasional short discontinuous cracks not filled with epoxy | | | |
| 0.75 | 0.85 | | | | | |
| 1.00 | | 1.0 | Broken to retrieve core. Epoxy is visible on shear surface. Shear is through most, around some coarse aggregate. | | | Encountered rebar or conduit at 1.0 ft Drilling stopped. |
| 1.25 | | | See general description of concrete. | | | |

Hole No. DS-3

| DRILLING LOG | DIVISION North Central | INSTALLATION NCRED-G | SHEET OF SHEETS |
|---|---|---|---|

| | |
|---|---|
| 1. PROJECT Dam 20 | 10. SIZE AND TYPE OF BIT 4" Thin wall |
| 2. LOCATION (Coordinates or Station) Pier 27, Downstream Face | 11. DATUM FOR ELEVATION SHOWN (TBM or MSL) |
| 3. DRILLING AGENCY | 12. MANUFACTURER'S DESIGNATION OF DRILL Truco |
| 4. HOLE NO. (As shown on drawing title and file number) DS-3 | 13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED — UNDISTURBED |
| 5. NAME OF DRILLER Jerry Wickersham | 14. TOTAL NUMBER CORE BOXES |
| 6. DIRECTION OF HOLE ☐ VERTICAL ☐ INCLINED Horizontal DEG. FROM VERT. | 15. ELEVATION GROUND WATER |
| | 16. DATE HOLE — STARTED — COMPLETED Sep 87 |
| 7. THICKNESS OF OVERBURDEN | 17. ELEVATION TOP OF HOLE |
| 8. DEPTH DRILLED INTO ROCK | 18. TOTAL CORE RECOVERY FOR BORING % |
| 9. TOTAL DEPTH OF HOLE 1.1 | 19. SIGNATURE OF INSPECTOR |

| ELEVATION a | DEPTH b | LEGEND c | CLASSIFICATION OF MATERIALS (Description) d | % CORE RECOVERY e | BOX OR SAMPLE NO. f | REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g |
|---|---|---|---|---|---|---|
| 0.25<br>0.50<br>0.75<br>1.0<br>1.25 | 0.55<br>0.95 | 0.65<br>1.1 | Thin epoxy coating on surface. Concrete with 1½ inch maximum size natural gravel coarse aggregate. Paste is hard and dense with an occasional entrapped air void. Aggregates are hard and dense with an occasional fractured piece.<br>Angled crack from 0.55-0.65. Core is intact across crack. No epoxy noted in core | | | Compressive strength 6750 p.s.i.<br>Pulse velocity 12,578 f.p.s. |

Hole No. US-1

| DRILLING LOG | DIVISION North Central | INSTALLATION NCRED-G | SHEET OF SHEETS |
|---|---|---|---|
| 1. PROJECT Dam 20 | | 10. SIZE AND TYPE OF BIT 4" Thinwall | |
| 2. LOCATION (Coordinates or Station) Pier 27, Upstream Face | | 11. DATUM FOR ELEVATION SHOWN (TBM or MSL) | |
| 3. DRILLING AGENCY | | 12. MANUFACTURER'S DESIGNATION OF DRILL Truco | |
| 4. HOLE NO. (As shown on drawing title and file number) US-1 | | 13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN | DISTURBED / UNDISTURBED |
| 5. NAME OF DRILLER Jerry Wickersham | | 14. TOTAL NUMBER CORE BOXES | |
| 6. DIRECTION OF HOLE Horizontal ☐ VERTICAL ☐ INCLINED ____ DEG. FROM VERT. | | 15. ELEVATION GROUND WATER | |
| | | 16. DATE HOLE | STARTED / COMPLETED Sep 87 |
| 7. THICKNESS OF OVERBURDEN | | 17. ELEVATION TOP OF HOLE | |
| 8. DEPTH DRILLED INTO ROCK | | 18. TOTAL CORE RECOVERY FOR BORING % | |
| 9. TOTAL DEPTH OF HOLE 0.8' | | 19. SIGNATURE OF INSPECTOR | |

| ELEVATION a | DEPTH b | LEGEND c | CLASSIFICATION OF MATERIALS (Description) d | % CORE RECOVERY e | BOX OR SAMPLE NO. f | REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g |
|---|---|---|---|---|---|---|
| 0.25 | | | Thin epoxy coating on surface. Occasional discontinuos angled cracks passing around and through pieces of coarse aggregate. No epoxy noted in core. | | | Compressive Strength 5880 p.s.i. Pulse velocity 11,088 f.p.s. |
| 0.50 | | | | | | |
| 0.75 | 0.7 | 0.8 | Broken to retrieve core at 0.8 ft. Shear is through coarse aggregate. | | | |
| 1.00 | | | See general description of concrete | | | |

Hole No. MO-1

| DRILLING LOG | DIVISION North Central | INSTALLATION NCRED-G | SHEET OF SHEETS |
|---|---|---|---|
| 1. PROJECT Dam 20 | | 10. SIZE AND TYPE OF BIT 4" Thinwall | |
| 2. LOCATION (Coordinates or Station) Pier 27, Missouri Face | | 11. DATUM FOR ELEVATION SHOWN (TBM or MSL) | |
| 3. DRILLING AGENCY | | 12. MANUFACTURER'S DESIGNATION OF DRILL Truco | |
| 4. HOLE NO. (As shown on drawing title and file number) MO-1 | | 13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN | DISTURBED / UNDISTURBED |
| 5. NAME OF DRILLER Jerry Wickersham | | 14. TOTAL NUMBER CORE BOXES | |
| 6. DIRECTION OF HOLE Horizontal ☐ VERTICAL ☐ INCLINED ____ DEG. FROM VERT. | | 15. ELEVATION GROUND WATER | |
| | | 16. DATE HOLE | STARTED / COMPLETED Sep 87 |
| 7. THICKNESS OF OVERBURDEN | | 17. ELEVATION TOP OF HOLE | |
| 8. DEPTH DRILLED INTO ROCK | | 18. TOTAL CORE RECOVERY FOR BORING % | |
| 9. TOTAL DEPTH OF HOLE 0.7 | | 19. SIGNATURE OF INSPECTOR | |

| ELEVATION a | DEPTH b | LEGEND c | CLASSIFICATION OF MATERIALS (Description) d | % CORE RECOVERY e | BOX OR SAMPLE NO. f | REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g |
|---|---|---|---|---|---|---|
| 0.25 | | | Thin epoxy coating on surface. Concrete with 1½ inch maximum size natural gravel coarse aggregate and natural sand fine aggregate. Paste is hard and dense with an occasional entrapped air void. Aggregates are hard and dense with and occasional fractured piece. No epoxy noted in core. | | | Compressive Strength 7260 p.s.i. Pulse velocity 12,500 f.p.s. |
| 0.50 | 0.6 | | | | | |
| 0.75 | | 0.7 | | | | Encountered rebar or conduit at 0.8ft. Drilling stopped. |

Hole No. IL-1

| DRILLING LOG | DIVISION North Central | INSTALLATION NCRED-G | SHEET OF SHEETS |
|---|---|---|---|

| | |
|---|---|
| 1. PROJECT Dam 20 | 10. SIZE AND TYPE OF BIT 4" Thinwall |
| 2. LOCATION (Coordinates or Station) Pier 27, Illinois Face | 11. DATUM FOR ELEVATION SHOWN (TBM or MSL) |
| 3. DRILLING AGENCY | 12. MANUFACTURER'S DESIGNATION OF DRILL Truco |
| 4. HOLE NO. (As shown on drawing title and file number) IL-1 | 13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED — UNDISTURBED |
| 5. NAME OF DRILLER Jerry Wickersham | 14. TOTAL NUMBER CORE BOXES |
| 6. DIRECTION OF HOLE Horizontal ☐ VERTICAL ☐ INCLINED ___ DEG. FROM VERT. | 15. ELEVATION GROUND WATER |
| | 16. DATE HOLE — STARTED — COMPLETED Sep 87 |
| 7. THICKNESS OF OVERBURDEN | 17. ELEVATION TOP OF HOLE |
| 8. DEPTH DRILLED INTO ROCK | 18. TOTAL CORE RECOVERY FOR BORING % |
| 9. TOTAL DEPTH OF HOLE 0.6 | 19. SIGNATURE OF INSPECTOR |

| ELEVATION a | DEPTH b | LEGEND c | CLASSIFICATION OF MATERIALS (Description) d | % CORE RECOVERY e | BOX OR SAMPLE NO. f | REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g |
|---|---|---|---|---|---|---|
| 0.25 | | | Thin epoxy coating on surface. Several horizontal and angled cracks. One of the cracks is filled with epoxy, others have no epoxy. | | | |
| 0.50 | | | | | | |
| | 0.6 | | Broken to retrieve core. Shear is through coarse aggregate. No epoxy on shear surface. | | | |
| 0.75 | | | See general description of concrete. | | | |

Hole No. Top 1

| DRILLING LOG | DIVISION North Central | INSTALLATION NCRED-G | SHEET OF SHEETS |
|---|---|---|---|

1. PROJECT Dam 20
2. LOCATION (Coordinates or Station) Pier 27, Top, Upstream
3. DRILLING AGENCY
4. HOLE NO. (As shown on drawing title and file number) Top 1
5. NAME OF DRILLER Jerry Wickersham
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN
8. DEPTH DRILLED INTO ROCK
9. TOTAL DEPTH OF HOLE 1.8'
10. SIZE AND TYPE OF BIT 4" Thinwall
11. DATUM FOR ELEVATION SHOWN (TBM or MSL)
12. MANUFACTURER'S DESIGNATION OF DRILL Truco
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN | DISTURBED | UNDISTURBED
14. TOTAL NUMBER CORE BOXES
15. ELEVATION GROUND WATER
16. DATE HOLE | STARTED | COMPLETED Sep 87
17. ELEVATION TOP OF HOLE
18. TOTAL CORE RECOVERY FOR BORING %
19. SIGNATURE OF INSPECTOR

| ELEVATION a | DEPTH b | LEGEND c | CLASSIFICATION OF MATERIALS (Description) d | % CORE RECOVERY e | BOX OR SAMPLE NO. f | REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g |
|---|---|---|---|---|---|---|
| | | | Thin epoxy coating on surface. Vertical crack in core. Crack is .03 inches wide at top of core and narrows with depth to .003 inches at 1.1 ft. depth. Same crack on opposite side of core extends to 1.3'. Epoxy is visible on shear surface at 1.1 ft. | | | * Core taken directly over an injection port. |
| 0.25 | | | | | | |
| 0.50 | | | | | | |
| 0.75 | | | | | | |
| 1.00 | | | Shear is through coarse aggregate. Broken to retrieve core at 1.0 | | | |
| | 1.1 | | | | | |
| 1.25 | | | Vertical crack is completely filled with epoxy except near top surface. Epoxy may have drained away. | | | |
| 1.50 | | | | | | |
| | 1.6 | | | | | |
| 1.75 | | | | | | |
| | 1.8 | | Broken to retrieve core at 1.8 ft. Shear is through coarse aggregate | | | |
| 2.00 | | | See general description of concrete. | | | |

Hole No. Top 2

| DRILLING LOG | DIVISION North Central | INSTALLATION NCRED-G | SHEET OF SHEETS |
|---|---|---|---|

| | |
|---|---|
| 1. PROJECT Dam 20 | 10. SIZE AND TYPE OF BIT 4" Thinwall |
| 2. LOCATION (Coordinates or Station) Pier 27, Top, Downstream | 11. DATUM FOR ELEVATION SHOWN (TBM or MSL) |
| 3. DRILLING AGENCY | 12. MANUFACTURER'S DESIGNATION OF DRILL Truco |
| 4. HOLE NO. (As shown on drawing title and file number) Top 2 | 13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED — UNDISTURBED |
| 5. NAME OF DRILLER Jerry Wickersham | 14. TOTAL NUMBER CORE BOXES |
| | 15. ELEVATION GROUND WATER |
| 6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED ______ DEG. FROM VERT. | 16. DATE HOLE — STARTED — COMPLETED Sep 87 |
| 7. THICKNESS OF OVERBURDEN | 17. ELEVATION TOP OF HOLE |
| 8. DEPTH DRILLED INTO ROCK | 18. TOTAL CORE RECOVERY FOR BORING % |
| 9. TOTAL DEPTH OF HOLE 1.65 | 19. SIGNATURE OF INSPECTOR |

| ELEVATION a | DEPTH b | LEGEND c | CLASSIFICATION OF MATERIALS (Description) d | % CORE RECOVERY e | BOX OR SAMPLE NO. f | REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g |
|---|---|---|---|---|---|---|
| 0.25 | | grout | Thin epoxy coating on surface. Approximately 1/3 of area of core in top 0.5 ft. depth is grout pad. Several vertical cracks to about 0.7 ft. depth, filled with epoxy. | | | * Core taken directly over injection port. |
| 0.50 | 0.5 | | | | | |
| 0.75 | 0.85 | | Horizontal crack at 0.85 ft. depth. Core is intact across crack; no epoxy in crack. | | | |
| 1.00 | 1.0 | | Broken to retrieve core at 1.0 ft. Shear is through coarse aggregate. No epoxy noted on shear surface. Discontinuous horizontal crack at 1.25 ft. Core is intact across crack. | | | |
| 1.25 | | | | | | |
| 1.50 | 1.5 | | Broken to retrieve core at 1.5-1.65 ft. Shear is through coarse aggregate | | | |
| 1.75 | 1.65 | | See general description of concrete. | | | |

# APPENDIX B

# PETROGRAPHIC EXAMINATION OF CORE TOP-2

DEPARTMENT OF THE ARMY
MISSOURI RIVER DIVISION, CORPS OF ENGINEERS
DIVISION LABORATORY
OMAHA, NEBRASKA 68102



27 JAN 1988

Subject: Petrographic Examination of Concrete Core

Project: Dam No.20, Canton, MO.

Intended Use: Investigation

Source of Material: Four inch diameter core of epoxy injected concrete

Submitted by: Chief, Geotechnical Section, Engineering Division, Rock Island District

Date Sampled: ____________, Date Received: 21 October, 1987

Method of Test or Specification: CRD-C 57, CRD-C 127, CRD-C139

References: Rock Island District Letter Request No. NCR-IA-88-0007 dated 19 October 1987

## SAMPLE IDENTIFICATION

1. One four inch diameter core of deteriorated concrete from Dam No. 20 was submitted by Rock Island District to evaluate the efficacy of epoxy grout injection on the pre-existing fractures present in the concrete structure. The core is labeled Dam 20, Pier 27, Top 2 by Rock Island District.

## TEST METHOD

2. The concrete core was visually examined initially to determine overall concrete condition and to select zones for further analysis. The core was examined with a stereo- and petrographic microscope in accordance with CRD-C 57, -C 127, -C 139 to determine the cause of concrete deterioration. The concrete was also examined using fluorescent light microscopy to identify and characterize the epoxy grout used to cement the numerous fractures present in the concrete. An unconfined compressive strength test was performed on the upper portion of the epoxy injected concrete to determine the degree of bonding between the epoxy and the concrete fracture surfaces.

## DISCUSSION

3. Petrographic examination of the 4 inch diameter concrete core indicates the concrete fracturing had been produced by both chemical and freeze - thaw deterioration. The concrete core contain well developed fractures that exhibit minor chemical alteration in the form of paste carbonation that does not penetrate below the fracture surfaces. The concrete outside the fracture zones is generally well constructed and of good quality. The core top is ir
MRD Lab No. 88/172

Buss/bh/3212

regular in shape and contains the exposed outer concrete surface and an irregularly shaped, *1/8 inch thick layer of epoxy cement that* appears to be well bonded to the underlying concrete . Grain mount analysis indicate the concrete is composed of an angular to well rounded, multicolored, crushed glacial coarse aggregate with a maximum diameter of approximately 1 1/4 inches. The coarse aggregate is composed of rock of diverse mineralogy that include sedimentary, meta - volcanic, igneous and metamorphic varieties. The fine aggregate is composed of a natural quartzose sand. The concrete paste is composed of portland cement that appears to be of normal hydration and of good quality. Deleterious constituents in the coarse aggregate consist of absorptive clay ironstone and weathered chert that are susceptible to freeze -thaw action ( Figure No. 3 ). Many of the chert and meta-volcanic rock particles are alkali-silica reactive and contain well developed reaction rims and are coated with copious amounts white silica gel precipitate ( Figure Nos. 8 and 9 ). These varieties of deleterious aggregate exhibits severe internal fracturing that extends out into the concrete paste forming an extensive fracture network. Fracturing is more pronounced in the upper 1.0 ft. of the concrete core with fractures less common from 1.0 ft. to the bottom of core at 1.65 ft. Two large open fractures are situated 0.88 and 1.0 ft. down from, and oriented sub-parallel to, the core top. These fractures pass through highly fractured deleterious coarse aggregate and are coated with substantial amounts of silica gel precipitate ( Figure Nos.1, 8 and 9 ). Entrapped and entrained air voids are not abundant and the concrete does not appear to be adequately air-entrained to protect against freeze - thaw action. The air voids present in the concrete are generally devoid of secondary mineralization although minor amounts of calcium carbonate and silica gel were identified coating some void interiors. A portion of core from 0 to 0.5 ft. contains a fine aggregate mortar delineated from the remainder of the concrete by a well defined vertical cold joint filled with epoxy cement ( Figure No. 5 ). This mortar appears to be of good quality with adequate air entrainment and lack of fracturing. Fracture analysis and measurements using fluorescent light microscopy ( Figure Nos. 6 and 7 ) indicate the epoxy has effectively filled approximately 77 % of the fractures to a point 0.84 ft. below the top of concrete. However, there is no evidence of epoxy cement on any fracture surfaces below this point. The epoxy has filled fractures ranging in width from 1.25 mm to 0.09 mm and fractures smaller than 0.09 mm did not appear to contain epoxy cement. Many of the fractures not filled by the epoxy in the upper 0.84 ft. of concrete were not interconnected to the fracture system. Thus, pathways were not available for the epoxy to reach these isolated fractures. An unconfined compressive strength test was performed on a portion of epoxy injected core at a depth interval of 0.09 ft. to 0.78 ft. below top of concrete. The unconconfined compressive strength was determined to be 4,775 psi. Examination of the test specimen after failure revealed the epoxy cement bond on fractures that were oriented parallel to subparallel to the core axis failed whereas cemented fractures perpendicular to the core axis remained intact inferring that epoxy bond strength approximates the compressive strength of the core. Since the concrete represented in this test specimen was extensively fractured and of apparent low strength before epoxy injection, the compressive strength of the test specimen after epoxy injection will be more representative of the bonding strength of the epoxy cement to the fracture surfaces than the compressive strength of the concrete.

CONCLUSION

4. Examination of fractured concrete injected with an epoxy grout from Dam No. 20 , Canton, MO. indicates the initial fracturing to be from distress generated by freeze thaw action and alkali - silica reactivity on deleterious clay ironstone and chert coarse aggregate particles. The deteriorated concrete had been repaired by injection of an epoxy cement along the fracture surfaces. The injection technique appears to have adequately impregnated the concrete only to a depth of 0.84 ft., below which there is no evidence of epoxy cement present on fracture surfaces. A portion of the epoxy injected concrete from 0.09 ft. to 0.78 ft. has a relatively high unconfined compressive strength of 4,775 psi, indicating the epoxy has provided a strong bond to the fracture surfaces appreciably increasing the strength of an otherwise weak, highly fractured concrete. It is recommended that the depth to which the concrete structure has been fractured be determined and the epoxy injection technique be modified to allow the impregnation the of fractures situated deeper in the concrete structure. To better evaluate the efficacy of future epoxy injection programs it is recommended that core samples be taken both prior to and after epoxy injection.

Submitted by:

R. K. SCHLENKER, P.E.
Director, MRD Laboratory

Buss/bh/3212

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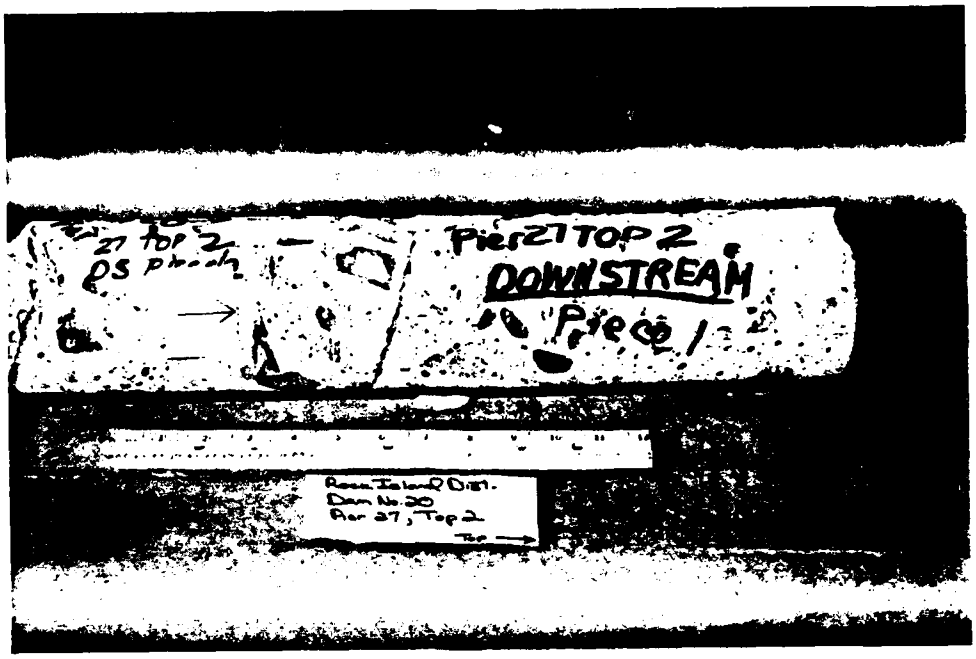
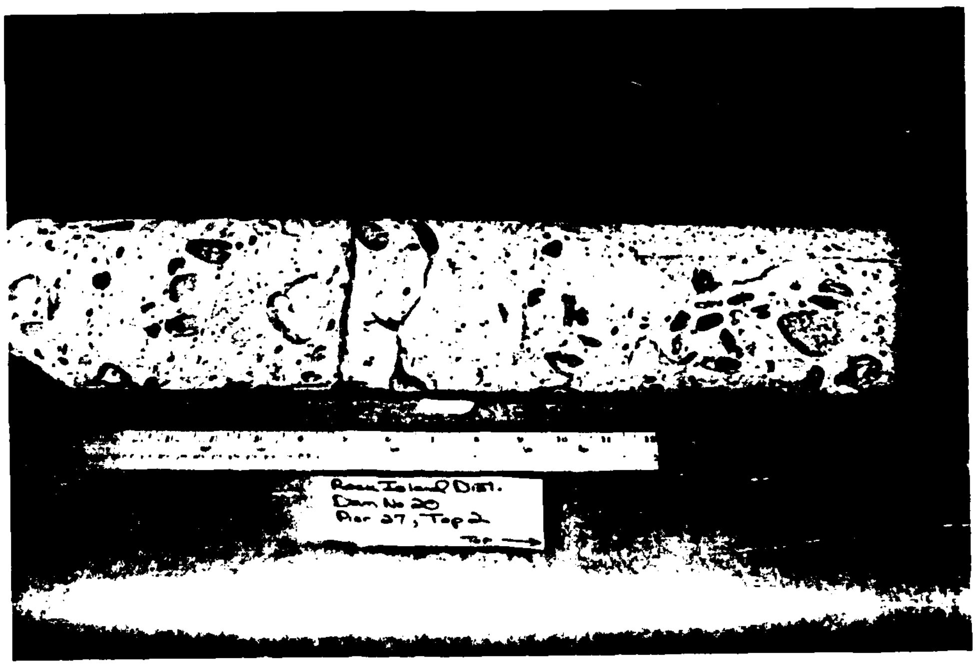
Missouri River Division, Corps of Engineers
Division Laboratory
Omaha, Nebraska

Figure No. 1




Side view of 4 in. dia. epoxy impregnated concrete core. Arrow denotes highly fractured deleterious chert coarse aggregate particle transected by well developed open frac -ture.

Figure No. 2




Same concrete core as in Figure 1, viewing opposite side. Note the two open fractures midway in the core at 0.88 ft. and 1.0 ft. below top of concrete.

Plate No. 1

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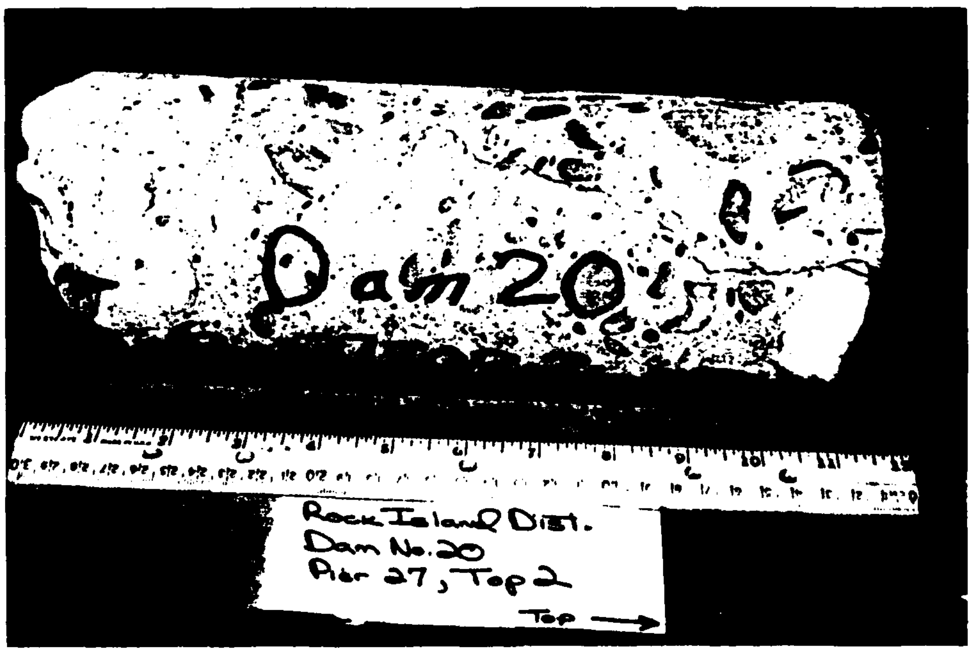
Figure No. 3

Same view as in Figure 2, arrows denote highly fractured freeze - thaw susceptible clay ironstone and chert coarse aggregate particles.

Figure No. 4




Same view as in Figure No. 1, close-up view of upper portion of epoxy injected concrete. Note prominent epoxy injected fracture oriented subparallel to the core axis.



Plate No. 2

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Figure No. 5


Magnified view of upper portion of concrete core shown in Figure 3. Note the light gray white chert coarse aggregate that have been highly fractured by freeze - thaw action.

Figure No. 6

Same view as in Figure 5 as viewed in fluorescent light. Epoxy cemented fractures stand out as luminous, irregularly shaped lines on the core surface.

Plate No. 3

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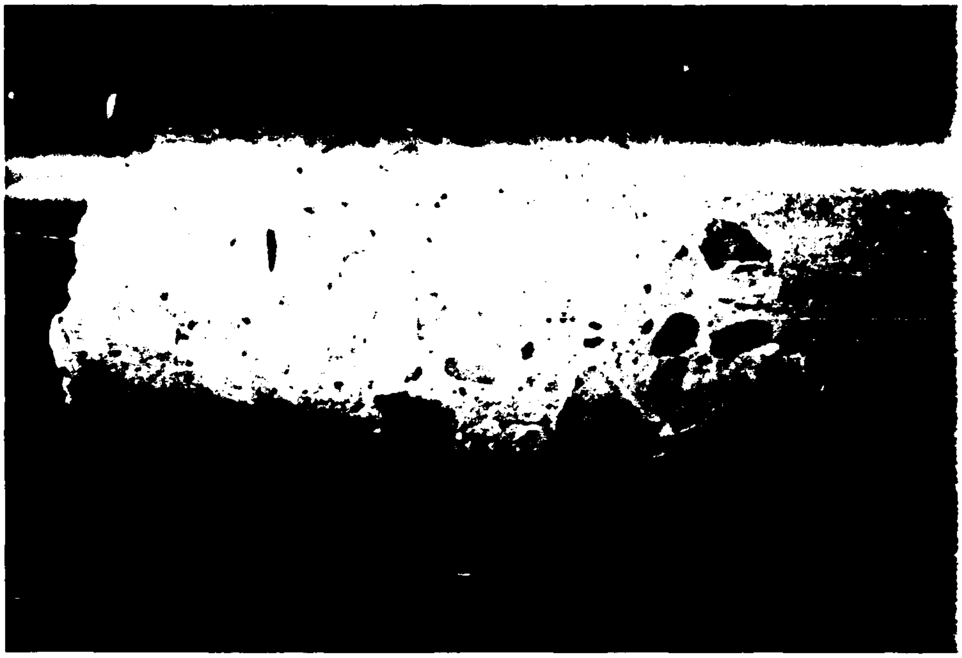
Figure No. 7



Same view as in Figure No. 5 , close-up view of lower portion of core. Open fracture forms the core end at 0.88 ft. below top of core.

Figure No. 8

View of lower fracture surface located 0.88 ft. below top of core. Note alkali - silica reaction rims in deleterious coarse aggregate particles and white silica gel precipitate coating fracture surfaces.

Plate No. 4

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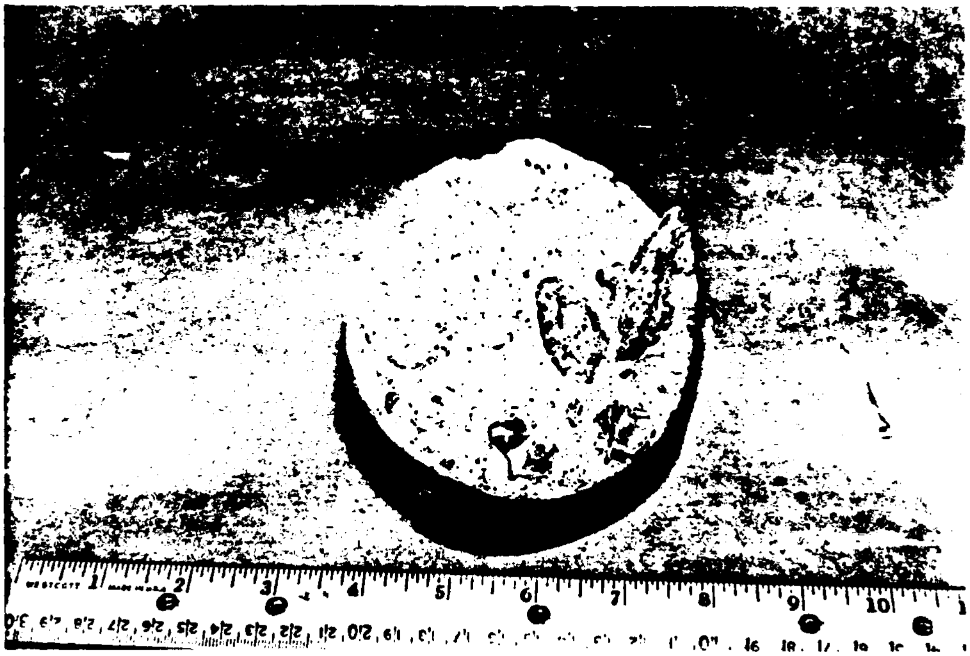
Figure No. 9



View of upper fracture surface located 1.0 ft. below top of concrete. Note white silica - gel soaking concrete paste surrounding alkali - silica reactive coarse aggregate.



Plate No. 5